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MEMBERSHIP

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS

A PROBLEM OF SOIL IN TRANSPORTATION IN THE COLORADO RIVER

By S. L. Rothery,[1] M. Am. Soc. C. E.

Synopsis

This paper is concerned principally with the problem of excluding soil, transported by the Colorado River, from the Canal System of Imperial Valley, in California. Its purpose is: (*a*) To feature the relative importance of the bed load of the Colorado River and to provide some conception of the extent of the unknown volumes of soil transported; (*b*) to present known and anticipated future river-flow conditions pertinent to soil conveyance; and (*c*) to propose fundamental requirements for a diversion structure that will exclude all of the bed load and a part of the suspended load, thus permitting minimum sizes for—or, perhaps, eliminating as unnecessary—the enormous settling basins which are expected to desilt the large diverted flow, and also permitting lessened sluicing operations for the disposal of the sludge.

The publication of specific research and a pooling of available knowledge before a design for such an important diversion is finally adopted, may suggest a solution for the automatic exclusion of much of the soil load. Each 1 000 000 cu. yd. of soil that is prevented from crossing the sill of the head-gate, can represent an annual saving of tens of thousands of dollars if relief must be obtained by dredging. With 10 000 000 cu. yd. of soil automatically excluded, such savings are probably in the hundreds of thousands of dollars, depending on the remedial measures that have to be taken when such large volumes of transported soil are continually arriving on the down-stream side of the diversion gates. In part, these savings are represented in the form of lessened maintenance operations throughout a vast canal system, in channels and sinks for waste-water disposal, as well as in the ditches of the individual ranches.

Note.—Discussion on this paper will be closed in March, 1933, *Proceedings*.

[1] Civ. and Cons. Engr., Los Angeles, Calif.

Introduction

As a background for a thorough understanding of the problem reference is here made to a paper on the subject by C. E. Grunsky, Past-President, Am. Soc. C. E., published in 1929.[2] As they apply to the intended purpose of this paper, the following items of information from that paper are summarized:

(1) The water supports and carries its suspended load throughout the entire length of the Imperial Canal. Sometimes there is less, and sometimes a little more, sediment in suspension at the head of the canal than at points 20 to 40 miles below but, generally, when the water is extremely muddy at the head it stays muddy throughout the entire length of the canal.[3]

(2) A phenomenon that illustrates the occasional conditions favoring the temporary conversion of the bed load of a river into a suspended load is the formation of a standing wave. This is noted in streams of moderate depth, flowing in beds with fine sand bottoms. It is a common phenomenon on the Lower Colorado River.[3]

(3) In 1918, the head-works of the canal were modified, suction dredges were installed, and, in a period of 30 months, nearly 15 000 000 cu. yd. of material was excavated. While it is not quite clear what effect this had on the lower reaches of the Imperial Canal, it is certain that if it had been allowed to remain in the canal, it would have traveled down stream to the annoyance of the Canal Management and the farmer.[4]

(4) The dredging did not relieve the canal of its bed load entirely. The dredged sumps in the head of the canal did not trap all the silt that came over the flash-boards of the Rockwood Gate. Even 500 000 cu. yd. per month does not represent the entire load of fine sand carried as a bed load by the canal. How much more was being transported is not known, and no entirely satisfactory basis for estimating it has been found.[4]

(5) Assuming that there must have been some débris and other material held in suspension temporarily during ordinary flow conditions, the bed load of the Colorado River (with its annual discharge of about 15 000 000 acre-ft. of water at Yuma) should be about six or seven times as great as that of the canal, or 25 000 to 30 000 acre-ft. per year.[4]

(6) Observations at Yuma in 1914 indicated that when the Colorado River was heavily charged with silt, the settling was nearly always slower than when the silt load was light. The Imperial Canal does not hold quite as much silt in suspension as the river. The water in the canal is less turbulent and the heavier grains of silt drop down and become part of the bed load.[5]

(7) Turbidity in the river varies with depth. The increase of the suspended load downward from the surface bears no fixed relation to the loads at the surface or at mid-depth. The increase with depth is nearly constant in

[2] "Silt Transportation by Sacramento and Colorado Rivers and by the Imperial Canal," by C. E. Grunsky, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1104.

[3] *Loc. cit.*, p. 1113.

[4] *Loc. cit.*, p. 1117.

[5] *Loc. cit.*, p. 1122.

the sense of being independent of the total suspended load, the increase of percentage being 0.11 from the surface to the bottom. In other words, if the percentage by weight at the surface is represented by P, that at the bottom is $P + 0.10$.[6]

(8) The Colorado River is not as muddy at low stages as at moderately high stages, as, for example, a discharge of 15 000 to 50 000 sec-ft. The river is muddiest when it is discharging 15 000 to 30 000 cu. ft. per sec. Turbidity is highest in the early spring and in the fall.[7]

(9) Perhaps there are other causes affecting the degree of muddiness. For example, the silt in the lower river shows many signs of being dispersed, that is, the colloidal particles seem to be carrying negative charges. Possibly, the source of the water and its chemical composition affect the degree of dispersion, or of flocculency, as the case may be, and, therefore, cause a greater or smaller volume of silt in suspension. When clays are dispersed, each flocculated particle splits into many subdivisions, sometimes so small as to be colloidal.[8]

(10) Table 11[9] of Mr. Grunsky's paper shows that in periods, such as October to December, 1925, and January, February, June, and July, 1926, the suspended load increases with the distance below the Grand Canyon; in other periods, it remains fairly constant. In September, 1926, there was even a decrease of suspended load. These results seem to confirm the conclusion that, at times, the river picks up material from its bed and, at other times, it drops part of its suspended load.[10]

(11) The facts illustrated in Mr. Grunsky's Fig. 6[8] give no indication that can be used in estimating the bed load of the river, except possibly that a considerable portion of it originates below the Grand Canyon.[11]

The Bed Load

An average of 15 000 acre-ft. of silt, the equivalent of 24 200 000 cu. yd., has been diverted annually in suspension into the Canal System of Imperial Valley, California. In 1923, the peak quantity was 24 900 acre-ft., or 40 172 000 cu. yd. In addition to this quantity, large volumes of bed sand and coarse silt, estimated at more than 4 000 acre-ft.,[4] or 6 453 000 cu. yd., have been transported as the annual bed load along the Main Canal. The respective annual quantities transported on the Lower Colorado River are estimated to be: 94 000 to 111 000 acre-ft. of suspended soil, and 20 000 to 30 000 acre-ft. of bed load.[8] These estimates of the bed load are believed to be much too small.

At a great diversion works, 19 miles up stream from Yuma, Ariz., provision is being made (1932) for a maximum inflow into a main canal of two to two and one-half times that of the present summer irrigation demand. Apparently, this should make possible, the doubling of each of the foregoing

[6] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1127.
[7] *Loc. cit.*, p. 1129.
[8] *Loc. cit.*, p. 1130.
[9] *Loc. cit.*, p. 1131.
[10] *Loc. cit.*, p. 1132.
[11] *Loc. cit.*, p. 1133.

two soil quantities in the main canal if the intercepting influence of the huge reservoir formed by Hoover Dam could be excepted. A cost of $3 375 000 is listed tentatively for the construction of desilting works at the proposed site.

That the suspended load of the Colorado River, as now accepted,[12] is only a part of the total soil annually transported, is logically shown by the following reasoning applicable to the spring flood each year, as well as in a lesser degree when smaller floods or freshets occur. Consider, for example, the 300-mile stretch up stream from Yuma. The bed is filled with soil subsequent to several winter months of low-river stages; then 4 to 6 weeks of increasing flow to the flood peak in May or June completely removes a solidity of soil represented by the product of 300 miles, an average scoured depth of, say, 9 ft., and a river width of, say, 600 ft. Any material in this space is influenced by fluid velocities of 4 to 10 ft. per sec., and it is, therefore, impossible for deposition or up-stream travel of any sand or silt particles to refill the scoured prism before the discharge is in the decreasing stages. This product is equivalent to 316 800 000 cu. yd., or 196 400 acre-ft. of soil completely removed and transported hundreds of miles in a period of only a few weeks. On the other hand, the estimated soil volume carried in suspension averages about one-half this quantity for an entire year!

The scoured deepening is often more than 20 ft. At Yuma in 1907 and, again, in 1909, it was found that for an increase in the gauge height the bed was lowered approximately 30 ft.[13] River widths of flood stages are between 800 and 1 200 ft., and the time of travel from the Grand Canyon to Yuma is about 4 days in high-water stages. Therefore, the values given for the elongated prism of removed soil are conservative, and the entire length is at a scoured maximum simultaneously during the few days of peak flow.

The additional square feet of scoured area of cross-section probably cannot be expressed in a percentage of the water volume as solid material in motion,

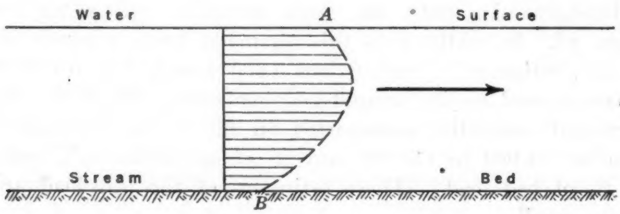

FIG. 1.

since the distance to deposition and the time unit of the flood duration do not seem to permit of a constancy of expression.

The velocity curve of a stream in a vertical plane is represented by the line, *A B*, in Fig. 1, the maximum velocity being in the upper half of the vertical depth, and the minimum at the stream bed. The fine silt particles that aggregate the "suspended" soil load, and that are responsible for the turbidity

[12] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1142.
[13] *Loc cit.*, Vol. LXXVI (1913), p. 1214.

of the stream, are buoyed up and transported in the upper two-thirds to three-fourths of the stream depth with less effort on the part of the stream, than in the lower depth, while general uniformity of flow is maintained, as is the case of an irrigation canal whether it is 1 mile or 300 miles in length.

Particles too heavy to rise, or too granular to be retained in suspension (unless turbulence is produced by some cause to destroy uniformity of flow) are transported by rolling and saltation along the bed by the lesser velocities, which, however, are not much less than those above the center depth. The velocity of flow necessary to move fine sand is 0.70 ft. per sec., and, for coarse sand, 0.80 to 1.00 ft. per sec.[14] The mean velocities for flow conditions in the channels and canals of the Colorado Delta region are 3 to 6 ft. per sec.; so that with bed velocities known to be greater than 1.00 ft. per sec. in the more slowly moving canal water, when flood conditions exist in the river the travel of the bed load is both rapid and continuous.

Some samples of the traveling bed load were collected at Grand Canyon from 1925 to 1928. When the sampling bottles were near the bottom with the mouths pointed up stream the silt content was 35% greater for a mean of sixteen samples, than when the samples were taken in the usual upright position. For six samples taken in a similar manner, but still nearer the bottom, the silt content was 40% greater.[15] Such increased percentages are properly part of the unknown bed load, if only for the reason that they have never been considered in estimating the suspended loads. This also shows that there is a traveling slurry of soil contiguous to the bed at normal river stages.

Consider such a fluid slurry in the lowest, say, 3-ft. depth of a large flood. Bed sand itself has no cohesive qualities, but it can have compactness when the interstices are filled with fine silt and when there is a weight due to depth of water above. The drag of increasingly scurrying velocities over it frets away the compacted particles, giving them initial movement and lessened weight due to becoming freely submerged. The loosened grains soon attain momentum, the velocity of which approximates that of the fluid influence surrounding them. This surely must be true for sand, since egg-sized pebbles are in motion with 3-ft. velocities, and boulders, 6 to 8 in. in diameter, are moved when velocities of 5 to 6 ft. per sec. are attained, and this latter rate is only the average mean velocity of Colorado floods.

If the sand load thus being shot along in the bottom 2 or 3 ft. of the flood, were represented only by a 3-in. average thickness of solidity for a 600-ft. width of river and averaging a velocity at the bottom of only 3 ft. per sec., the bed-load volume would be 17 cu. yd. per sec. at any given point, which is equivalent to 1 440 000 cu. yd. per day, or 60 000 000 cu. yd. in the 6 weeks of rising flood. Visualize this soil supply as coming from not any one given location, but from the continued welling up of the bed into fast fluid motion along hundreds of miles of this elongated soil prism. The 3-ft. velocity represents, say, 50 miles per day. Then, in a 300-mile length there would be a

[14] "The Control of Water," by the late Philip A' Morley Parker, M. Am Soc. C. E., p. 495.

[15] *Water Supply Paper 636B*, U. S. Geological Survey, p. 26.

daily feeding of sand to the traveling load of 1 440 000 × 300 ÷ 50 = 8 640 000 cu. yd., which, if continuous for 6 weeks, or 42 days, becomes 360 000 000 cu. yd., or 223 000 acre-ft. of soil.

In the first flood stages, at 30 000 sec-ft., the traveling thickness of equivalent solidity may be, say, less than an inch, and at the flood peak it may be the equivalent of feet in thickness. Any value used must necessarily be an assumption. The writer is of the opinion that 3 in. as an average is too conservative, and that scour is a continuous process of excavation in a great depth of alluvium, until flood velocities have passed. If the discharge lessens temporarily, the water surface lowers; but scour and the rate at which sand travels are functions of the flood velocities which are maintained for some days after the flood peak has passed.

Assuming that the bed of the river is of alternate "deeps" and "shoals," so that it may be claimed that the effective length is reduced 50%, which would divide this great scoured volume in two, the result would still be greater than that of the estimated suspended load per year. This soil movement has been accomplished in six weeks from only a part of the channel.

A conspicuous example of the power of the river to excavate soil was described in a previous paper[16] wherein the writer stressed the transportation of such soil volumes in a unit length of 100 miles.

Anticipated River Flow Conditions Below Hoover Reservoir

The average annual run-off of the Colorado River is 15 000 000 acre-ft., with years of maximum precipitation yielding more than 20 000 000 acre-ft. The planned storage capacity of Hoover Reservoir is 25 000 000 acre-ft. Until large agricultural uses are developed up stream from the reservoir to consume the water, the river bed below the huge dam, in consecutive wet years, may have to take care of a flow nearly equal to the annual run-off. A regulated continuous flow of 21 000 sec-ft. throughout the year would provide this requirement; but, because of the necessity of conserving head at the dam for development of power, such steady flow will never occur, and the regulation of discharge will depend upon the down-stream uses, and upon the estimated up-stream flood inflows as to their effect on the surface elevation of the conserved water in the reservoir.

Down-stream uses are a minimum in the winter months. With a release of 15 000 sec-ft. for six months to provide these uses, including the quantity required for sluicing at the Imperial Diversion, 5 500 000 acre-ft. will have been discharged from storage by March, and this volume in a wet year may have been more than regained by inflow, so that the reservoir has been maintained as nearly full. The spring flood with 9 000 000 to 10 000 000 acre-ft. in 60 days (as in May and June, 1917, 1920, and 1921) would demand a release of 83 000 sec-ft. While this discharge will undoubtedly be reduced in the demand for regulated down-stream control, it is to be expected that the range of flow will be between 15 000 and 65 000 sec-ft. in the lower river after the Hoover Reservoir is filled.

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[16] *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1412; also, *Engineering News-Record*, Vol. 95, (1925), p. 1068.

The stream will be a different and chastened river, however, when the Hoover Dam is placed in operation. Although the river issues from the dam as clear water without its up-stream soil loads, 300 to 450 miles of soil-filled river bed are ahead; but its capacity for deep scour and for refilling with soil its channelized bed will have been greatly curtailed.

Since the profile gradients have resulted from the necessary velocities for silt-laden water in the alluvium, they will not be stable enough to produce and maintain the lesser velocities needed for the flow of clear water. The required gradients will be less steep and the stream, in striving to attain these new slopes, will continue to pick up soil loads, because the moderately high river stages are capable of transporting a maximum turbidity. The references (Items (8), (10), and (11)), to highest soil loads for discharges of 30 000 to 50 000 sec-ft., have their verification in studies by Raymond A. Hill, M. Am. Soc. C. E., from 3 000 samples taken between 1908 and 1916 at Yuma.[17] Mr. Hill calls attention to the fact that the characteristics of a desilted river cannot be based on data acquired when the river was heavily laden with silt and that no one can predict which rate of discharge from a flood-control reservoir will produce the most stable condition.

In commenting on the maximum silt content at discharges of 40 000 sec-ft., E. W. Lane, M. Am. Soc. C. E., states[18] that the redistribution of flow might even increase the silt removal to more than 80 000 acre-ft. per year.

Eventually (after several years), the required river gradients for clear water flow will become established through gradual recession working up stream from control locations in the bed, such as from the diversion dams of the Imperial Intake and of the Metropolitan Water District at Parker, Ariz.; and also from locations where rock or hard soil is close to the river bed. With varying discharges (the maximum being four to five times as great as the minimum), erosion, deposition, and meandering will still occur, although to a much lessened extent than with the extremes of flow of the unregulated river.

Tributary streams, notably the Williams River, will frequently contribute sudden freshets transporting coarse soils and sand, because the declivities of these side creeks and washes are relatively great. That of the lower end of the Williams River is more than twice the stream gradient of the Colorado for a distance of 10 miles from their confluence, and the high discharges are given as 7 000 to 9 000 sec-ft.[19] On August 5, 1931, a cloudburst on the Williams water-shed produced an increase of discharge from 7 000 to 45 000 sec-ft. over night.[20]

It is concluded, therefore, that both suspended and bed loads will be present in the curtailed river flows below the Hoover Reservoir. The suspended loads, however, are expected to decrease in extent to almost negligible quantities as the regimen of the stream becomes stabilized. Bed sands are

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[17] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 419.
[18] *Loc. cit.*, p. 418.
[19] *Water Supply Paper 409*, U. S. Geological Survey.
[20] *Los Angeles Times*, August 7, 1931.

expected to be present always, although in much lessened quantities when the channel has attained equilibrium.

The physical difference of the "suspended" silts and the "bed" silts has been described[21] concisely by Harry F. Blaney, Assoc. M. Am. Soc. C. E. He distinguishes between the mechanical analyses given in Table 19 and in Table 1 of Mr. Grunsky's paper, previously mentioned, to the effect that 73% to 95% of the "suspended" load was fine enough to pass a No. 200 sieve, but that only 1.7% to 34% of the canal "bed" deposits will pass such a sieve in which the separation of the meshes is 0.10 mm. From these respective tables it is noted that the coarsest "suspended" silt passes a No. 40 sieve (wherein the meshes are 0.48 mm.), and the coarsest canal "bed" silt or sand passes a No. 20 sieve (wherein the meshes are 0.95 mm.).

The great bulk of the suspended load of the present turbid flows, therefore, is seen to be of the finest soil particles, but the proportion of such particles in the alluvial soils of the river bed, resulting from recurring deposits of each receding flood, is undoubtedly only a small proportion of the bed soils.

Charles Terzaghi, M. Am. Soc. C. E., refers to a property of silts as being scale-like and fragile (see, also Item (9)), always sinking in stilled water with the flat side horizontal, whereas sand grains are of rounded and angular shapes. His analyses[22] afford an explanation of what is perhaps the fundamental characteristic difference between the "suspended" and the "bed" loads of the Lower Colorado. It is readily conceivable that lamina and disk shapes are more sensitive to support and carriage in fluid motion than granular ones.

The sieve analyses mentioned, also show similarity of sizes of particles in both Tables 1 and 19 of Mr. Grunsky's paper previously noted, although the relative percentages of fineness and coarseness differ. As a result of silt research on the Nile River in Egypt,[23] "coarse sand" was classified as that collected on a No. 100 sieve; "fine sand" as that passing the No. 100 sieve, but caught on No. 200 sieve; and silt and clay as that passing the No. 200 sieve. The coarse sand in suspension in the bottom 3 ft. where the Nile is 26 ft. deep, is given by Mr. A. B. Buckley as 16 times the quantity of such sand in suspension in the surface 3 ft.; that for the fine sand as 7 times the respective quantity; while that for the suspended matter—silt and clay—shows little difference, the bottom quantity being 1.2 times that in the surface water. This is in close agreement with Mr. Grunsky's findings for the suspended silt content of the Colorado River (see Item (7)), which content is, however, ten times that of the Nile.[24]

Therefore, the clear-water flow from the Hoover Reservoir, in the erosive process of flattening the stream gradients, will absorb readily into suspension the finer silts to aggregate the capacity soil load. The granular sands and coarser silts will contribute in a lesser degree to this load, until such

[21] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1137.
[22] *Engineering News-Record*, Vol. 95, p. 912.
[23] *Minutes of Proceedings*, Inst. C. E., Vol. CCXVI, p. 205.
[24] *Loc. cit.*, p. 214.

suspensible matter, entrapped in the alluvial deposits, has been released and transported. In other words, the silts (and turbidity) will be eliminated from the bed soils of the channel first, leaving the sands to continue to move by rolling and saltation after the clearness of the stream has become established.

Furthermore, sand will always be present, more or less, because each year tributary streams and washes that have steep gradients, will add supplies of sand and coarse soils transported by the cloudbursts that are a climatic feature of the territory. The largest of these streams are the Williams River (which divides into the Big Sandy River and the Santa Maria River), Sacramento Wash, Tyso Wash, Bouse Wash, and Arroyo Seco. Again, with the decreased flows becoming more uniformly regulated in distant years, scoured channelized depths will cease to exist, and the curtailed stream will tend to assume a wide shallow cross-section that is virtually all bed (see Fig. 2, in which the greater distance (*a*) is the width of channel for a stream the bed

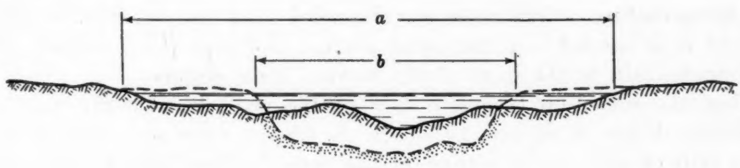

FIG. 2.

of which is in sandy soil, and the distance (*b*) is that width for an equivalent stream volume flowing in soils that have cohesive qualities, as in silts and clays). Because in such a stream, the relative area of the bed to the wetted surface is great, giving a large transporting width, sand travel may be expected to be a continuous feature of the flow; therefore, any structural provision effecting its exclusion from the diverted irrigation water will be permanently beneficial.

Structural Requirements for Exclusion of the Bed Load

The quantity of sand and bed silt that has been passing through the headgate into the Main Canal is unmeasured. How much more than 500 000 cu. yd. per month (see Item (4)) is being transported is not known. Undoubtedly, the actual quantity has been more than twice the original estimate of 500 000 cu. yd. based on suction dredger yardage. The sumps to which Mr. Grunsky refers[25] for trapping the sand, are deep excavations in the bed of a canal more than twice too wide for the arc limits described by the suction head of the dredge swinging about a lowered spud. These sumps cannot be made too close to the canal banks. The traveling bed load fills the deep excavations rapidly as they are being made over, say, 50% of the bed width, while traveling past the dredger over the other 50 per cent. Shut-downs for repairs and while the dredger is advancing to new locations, represent time lost, giving the advantage to the bed load, the travel of which is ceaseless.

[25] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1117.

Fig. 3 shows extreme low-river conditions on the river side of the intake gate, the bed of the river being flush with the lowest sill of the structure. The "drag" on the bed in the direction of the intake is apparent. The structure is composed of seventy-six openings that invite the traveling bed load, except during flood conditions. Fig. 4 shows the flood conditions, during which the bed is probably at great depth below the sill, which is also raised several feet; but the sudden retardation of velocity of the canal inflow, contributes the bed load in the canal.

When extremes of discharge are removed, with scoured river conditions the head-gate sill should have a sluicing velocity below it, tending to sweep the traveling bed load past the structure without coming over the sill; but with a diversion taking the canal supply from a direction at right angles to the river, interference of uniform flow conditions (especially when the draw-off is relatively large) is produced in the division of the water. The eddies and small whirlpool vortices thus formed, extend to some distance out stream from the structure, stirring the traveling bed load into suspension; whence, much of it is carried into the canal portion and over the head-gate sill, to be dropped again in the more slowly moving canal waters.

When the mixed-up turbulent flow of the river is suddenly quieted by the orderly diversion of, say, 10 000 cu. ft. of water per sec., there is a continued rain of soil to the bottom of the canal. When this is observed for 86 400 sec. each day, the structure responsible for it assumes a significance contrary to its intended purpose. Figuratively, the structure seems to reach down wilfully into the depths of the river to stir up and collect as much of the soil load as possible before it can escape, and place it in the canal on the down-stream side of the head-gate merely to cause annoyance and expense. Then the silt must be transferred back into the river again for final disposal, to prevent its becoming a menace throughout the canal system.

The writer is of the opinion that a side offtake without under-sluices is an effective selector of bed silt. The drag on the river bed is toward the structure; raising the sill will obstruct the travel of the bed load only temporarily. The relatively small pocket in front of it is filled quickly to form a ramp sloping up to the sill elevation and permitting the travel to continue into the canal. Lowering the sill again causes the entire ramp to move in, offsetting the short respite gained while the pocket was being filled. Such a sill is, therefore, completely ineffective, apart from the turbulence (produced by the division of stream flow), which causes eddies and vortices to lift some of the approaching bed load into suspension to pass it through the diversion gates.

The greatest source of trouble and expense to canal maintenance and operations is removed if the bed load can be excluded entirely, and some added cost to effect this purpose is money well spent beforehand. When "sand is running,"[26] that phenomenon is visible in the disturbed hydraulic gradient and water-surface agitation for a length of several hundred feet in a main canal. The traveling bed sand accumulates in excessive volumes to constrict

[26] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1113.

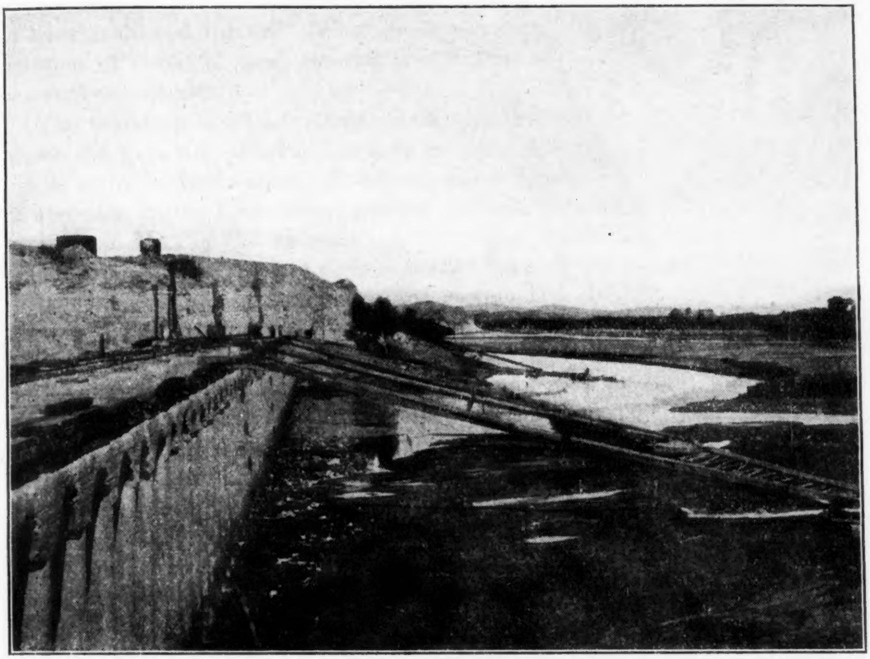

FIG. 3.—RIVER BED FLUSH WITH SILT FOR EXTREME LOW WATER AT ROCKWOOD INTAKE.

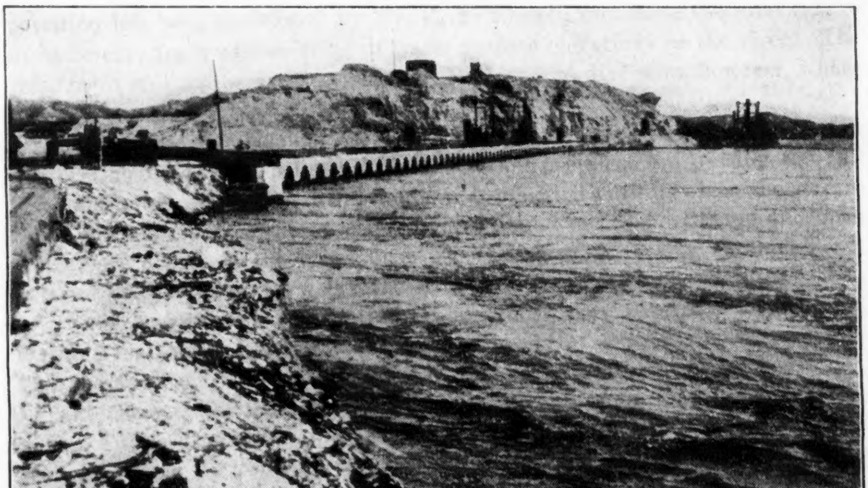

FIG. 4.—HIGH FLOOD AT ROCKWOOD INTAKE.

channelized flow in this canal, the uniform requirements of which are changed to whirlpools and turmoil. Whether or not there is surface evidence of the presence of blocks of sand, expense is involved for bank protection, dredging, or canal enlargement.

The traveling bed load should never pass through the diversion openings above the gate-sill. Furthermore, it is unnecessary to trap it in desilting basins or in dredged sumps. It must not be impeded or retarded in its travel of approach in the river before passing through the structure at an elevation lower than that of the gate-sill. A horizontal separation of the uniformly approaching flow to the structure is needed between the sill and a point some distance up stream, in order that the stream bed shall be kept beyond the influence of the contractural inflow through the diversion openings, thus preventing any disturbance on the approaching bed-load conditions. This can be effected by a light horizontal partition, supported on light walls, the upstream edge preferably having as much overhang as possible. No vortices or eddies can then penetrate to the traveling sand, which is disposed of through under-sluices without any slowing up or trapping by deposition to necessitate rehandling.

A diversion structure, placed across the river normal to the stream, provided with such a partition and under-sluices properly proportioned, will exclude, automatically, all the bed load and that part of the suspended load in the lower water that passes under the partition. Relative to the total suspended load this part is at least in the ratio of the depth below the partition to the full depth of the stream. The logical deduction for such a partition has been described[27] by Mr. F. Y. Elsden, including the final refinement to care for a varied range of water-surface elevations in the river. The refinement will not be necessary at the new Imperial diversion, however, where a fairly constant river height will be maintained.

A necessary requirement is that the flow of approach be in a smooth uniform channel. When the diversion and under-sluices are passing the full river discharge for long periods, this is not difficult to attain, since the stream tends to stabilize its own approach channel; but when the diversion and open sluices are a part of the river discharge (the surplus passing through floodgates or over an adjacent weir), the stream division, or the accelerated velocities which are produced if the weir is too short, may require a longitudinal training wall extending up stream in the river for some distance, to assure that the approaching bed load of the diversion portion is not disturbed.

The correct weir length that will produce a minimum scouring turbulence with the high discharges, and a minimum of shoaling or deposition with subsequent return to low-water flow, is determinable. Should a long training wall be the solution for uniform approach to the diversion openings, care in determining its location is necessary to avoid shoaling in the approach channel, when the major portion of the river flow is passing through the floodgates or over the weir.

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[27] *Water and Water Engineering*, October 20, 1923.

Figs. 5 and 6 show the diversion works as tentatively proposed, with the changes made to provide the features described herein as being necessary for the successful exclusion of the traveling bed load. The disposal of this soil through the diversion structure is preferably a continuous and unobstructed

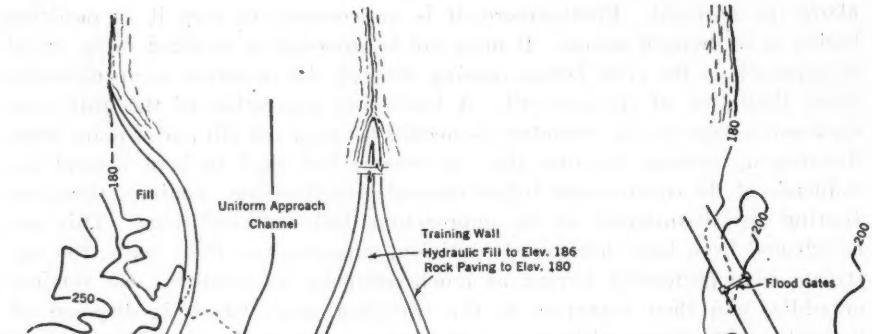

FIG. 5.—TENTATIVE PLAN OF DIVERSION WORKS.

transportation to a free discharge, which is regulated in volume by the sluice-gates, into the river bed on the down-stream side of the structure. The diversion openings, canal flume, and the under-sluices call for a frontage to the river of 750 ft. and a length of about 450 ft., as shown.

Figs. 7 and 8 show the works if desilting basins are to be used to obtain a partial deposition of the suspended load. The discharge from the under-sluices conveys the bed sand in a conduit that leads across under the lower side of the diversion gates, to empty through a gate-control well, into the river bed as before. Here, the works require a frontage of 1 200 to 1 250 ft. and a length of 1 550 ft. These illustrations are only diagrammatic.

The gates of the under-sluices may be closed to allow the bed load to accumulate. The partial deposition from the lower depth will then raise the bottom of the approach channel to the elevation of the diversion gate-sill, when the gates are re-opened to scour the bed down again. Thus, the under-sluices may be effective for either continuous or for intermittent sluicing

operations. A flash-board or raised sill, 1 ft. to 2 ft. high, at the diversion gate would provide further leeway for raising the bed of the approach channel before re-opening the sluices.

A trash rack placed up stream at the under-sluices would prevent submerged driftwood from entering the structure. A footwalk supported on light

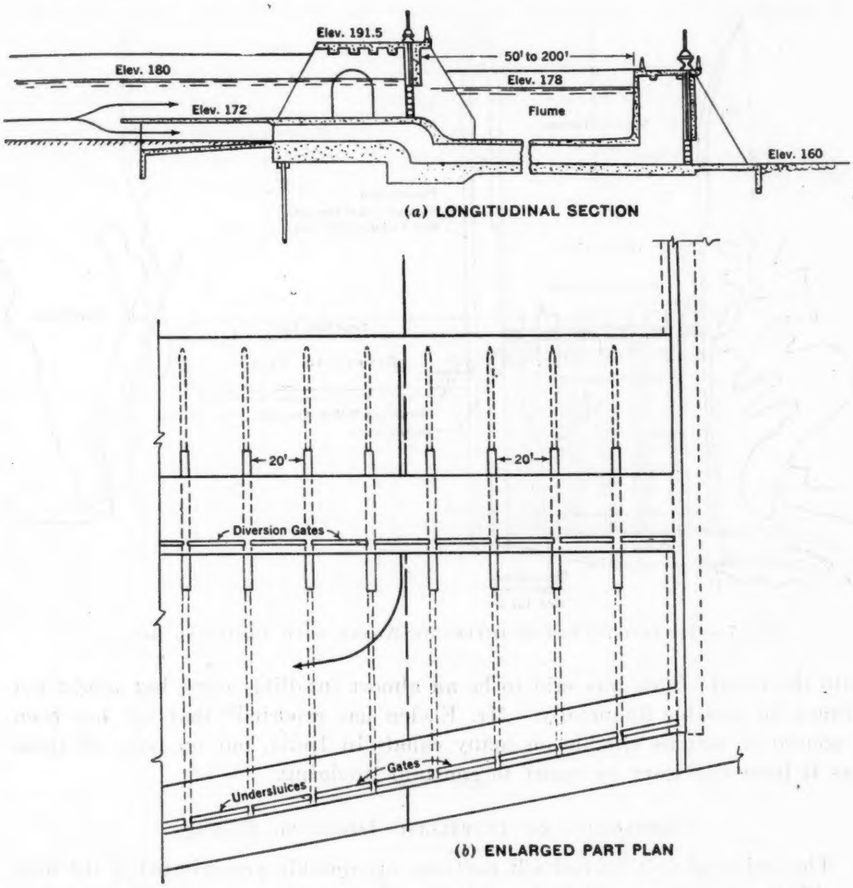

FIG. 6.—ENLARGED PLAN AND LONGITUDINAL SECTION OF DIVERSION WORKS.

piers rising from the horizontal partition at the under walls would furnish access for the removal of such driftwood; and flash-board grooves in these piers, with narrow openings through the horizontal partition to permit the lowering of the flash-boards, would be a means of closing the under-sluices in case of an emergency, or for inspection.

Mr. R. B. Buckley states[28] that the direction in which the gates travel when opening, is an important feature for the exclusion of bed sand. Refer-

[28] *Minutes of Proceedings,* Inst. C. E., Vol. CCXVI, pp. 215–216.

ring evidently to offtakes without under-sluices, he announced in 1922 that the difficulty due to silt accumulations at the heads of canals in India had been quite overcome. One canal was dredged every year for twenty years until it was discovered that if instead of the vents being opened from the bottom, they were opened from the top, the bed of the river was not scoured

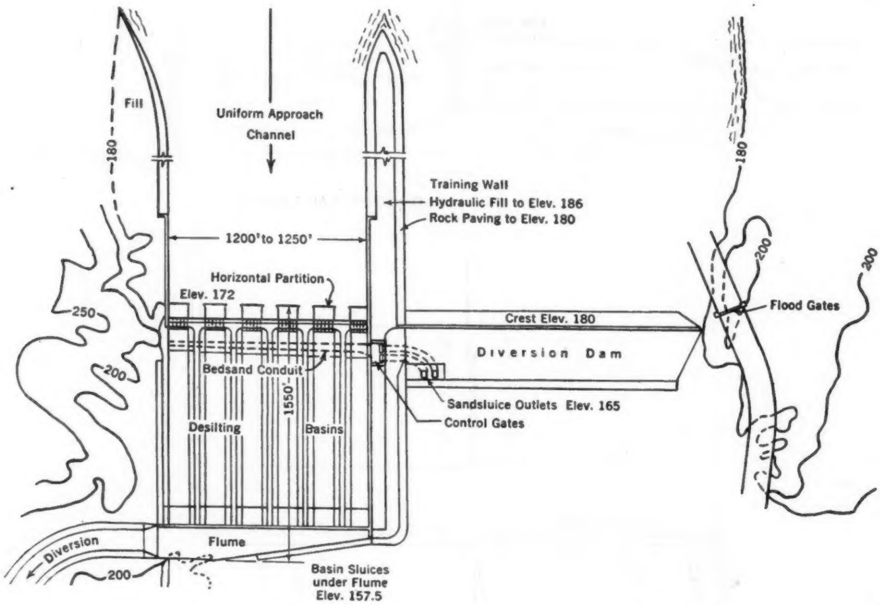

FIG. 7.—TENTATIVE PLAN OF DIVERSION WORKS WITH DESILTING BASINS.

into the canal. This was said to be an almost infallible cure, but might not always be possible financially. Mr. Elsden has reported[29] that silt has been a source of serious trouble on many canals in India, but on none of them has it been necessary to resort to constant dredging.

DESIRABILITY OF INSTALLING DESILTING BASINS

The bed sand and heavier silt particles are quickly precipitated if the flow is stilled or greatly retarded; but this is not so for the suspended load, which requires a period of detention in quiescent water for several hours.

The rate of deposition of soil in suspension in still water from the Nile River in Egypt, was found[30] to be 2 min. through a depth of 25 cm. for the sands which comprised 10% of the sediment, and 3 days for the silts and clays to settle through 50 cm. The rate for the Colorado River was found[31] to be 2 hours through a depth of 1 m., to the extent that a definite mud deposit is formed, leaving the water apparently clear to the eye if not closely inspected.

[29] *Minutes of Proceedings*, Vol. CCXVI, pp. 215–216, 225.
[30] *Loc. cit.*, Vol. CCXIX, p. 115.
[31] Seventh Biennial Rept., Dept. of Eng., State of California, p. 119.

If the flow in a long section of canal prism is stilled for sufficient time, the fine suspended silts settle slowly until they remain very lightly inter-supported as a loose sludge on the bed. The slightest movement of the water, such as the resumption of flow, causes the settlement to be disturbed and the silts, which are readily susceptible to diffusion in fluid motion, are again in suspension unless precautions are adopted to minimize the disturbance of

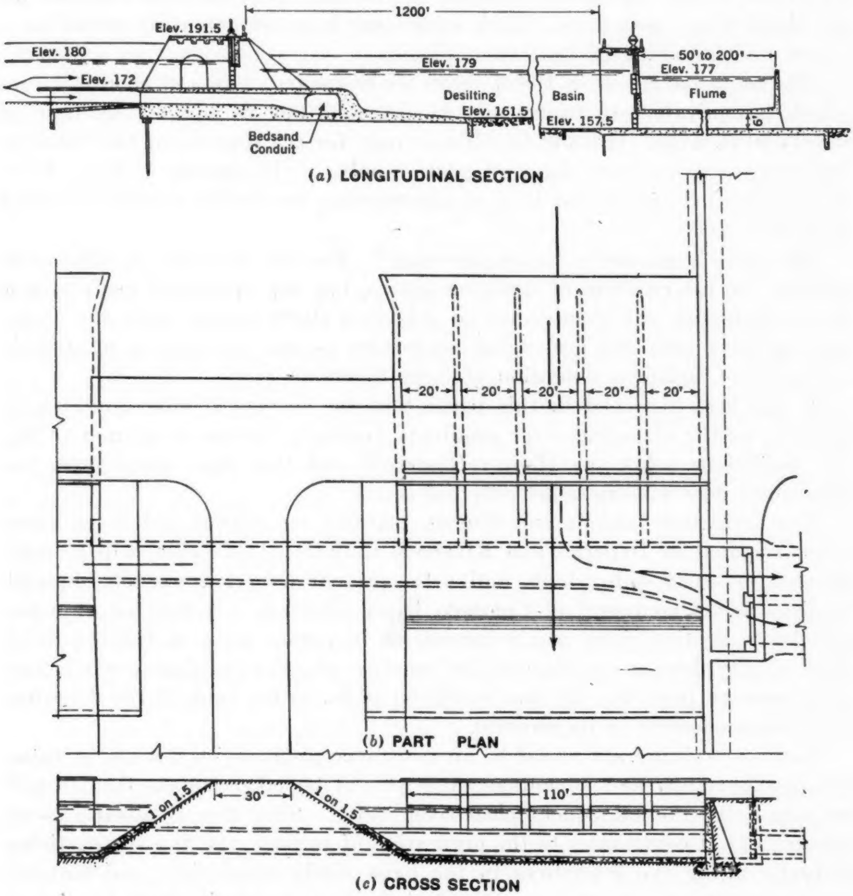

FIG. 8.—ENLARGED PLAN AND LONGITUDINAL SECTION OF DIVERSION WORKS WITH DESILTING BASINS.

this undeposited sludge. One such precaution would be to have the stilling basin relatively wide at right angles to the canal flow, drawing off the quieted surface water from a point on the wide side. The inflow to the basin should be directed toward those wide parts most remote from the position of the outflow.

Each 1 000 sec-ft. of flow stilled for 1 hour would require a basin capacity of 80 acre-ft., which is equivalent to a water area of 4 acres with a vertical

depth of 20 ft. If the maximum diversion of 15 000 sec-ft. is to be detained for 1 hour, a capacity of 1 200 acre-ft., or fifteen such basins are needed, with as many more providing the draw-off to maintain constant diversion flow. Thus, thirty such basins with a water area of 120 acres (requiring a total area of, say, 200 acres of ground) are necessary to obtain only 1 hour's detention of the flow, not even considering desludging operations and time required to fill the basins. A further increase of probably 50% would be required for the desludging operations. Each additional hour of detention would be a direct multiplication of the extent given.

The tentative layout in Fig. 7 shows six basins on an area of about 37 acres which is equivalent to twelve city blocks, each 300 ft. square, inclusive of streets 60 ft. wide. It could be effective only for the removal of the bed sand and coarse silts, by effecting a retardation only, of the velocity of flow. Even for this limited area a cost item of construction for desilting works is stated to be $3 375 000.

Are these great costly basins necessary? The bed load can be eliminated without the intervention of desilting basins, but the suspended loads cannot be so eliminated, and then decreased only in a slight degree, since the necessary desilting area and desludging operations assume too great a magnitude to permit of sufficient detention of large diversion flows.

It has been reasoned in this paper that the suspended silts diminish in quantity as the changing river gradients gradually become stabilized to the new conditions below the Hoover Reservoir, and that after some years the clear-water flow will transport only bed sand.

New irrigation laterals and ditches, required on 300 000 additional acres to be irrigated in Imperial and Riverside Counties, California, will be constructed in sandy soils which, unlike the present irrigated area of Imperial Valley, are not composed of Colorado River alluvium. It may be a double mistake to deprive these new watercourses in porous soils, of the beneficial effect of silty flow, and to deprive the ranchers who are reclaiming windswept sandy acreage from the silt that would be added to his fields if the desilting basins were effective in its removal.

Water containing silt would be an assistance proffered by Nature to these new developments, and advantage of it should be taken before the laterals begin to carry the subsequent clear-river water. Silty flow is advantageous because: (1) It contributes to the prevention of seepage, the fine silt particles gradually filling the interstices in the loose sandy canal sides and bottom, thus sealing these seepage passages; (2) it helps to maintain depth by giving stability to the side slopes of canals in sandy soils; (3) it will kill all aquatic growth by excluding light; and (4) it will give added surface soil which is also claimed to have a fertilizing value, to the rancher who has sandy land.

However, this added soil is of negative value to the long-suffering ranchers who farm the heavy alluvium of the present irrigated area; and impatience to obtain clear water is a factor that must be considered, if such water can be obtained even at high cost. A sudden conversion to clear-water flow will undoubtedly result in additional costs for canal maintenance, and this is an adverse factor to the change.

The arguments in this paper can be summarized as follows:

(*a*) The traveling bed load can be removed without desilting basins.

(*b*) The traveling suspended load cannot be greatly reduced with desilting basins, because the period of detention must necessarily be too long for precipitation of the fine silts.

(*c*) The suspended load will be a diminishing quantity with the stabilizing of new river gradients, and will not exist after a few years.

(*d*) Silty canal flow is desirable to new acreage during the years after the porous soils comprising the mesa lands are first reclaimed.

(*e*) Impatience and sentiment from the large agricultural communities of Imperial Valley may demand that all possible efforts be made to obtain clear-water flow in the canals, when the new diversion is effective.

Items (*a*), (*b*), (*c*), and (*d*) do not favor justification for 40 to 100 acres of huge desilting basins with large desludging structures, and subsequent daily sluicing operations. Item (*e*) may overrule the other four, in which case competent investigation, with some research, may be well worth while to determine factors governing the rapid desilting of fine silts, in order to obtain a minimum magnitude of works and of subsequent operations.

References have been made in Item (9) and in other places as to the colloidal phenomena in Colorado River water.[32] Particles of colloidal dimensions cannot be precipitated without the presence of an electrolyte, which may have to be added to cause flocculation. However, it is not believed that the term, "desilting," is intended to apply to this requirement for the purposes of clarifying irrigation water.

Conclusion

The assumptions used in this paper to estimate volumes are necessary to visualize the greatness of the transported loads. They are very conservative; even greater values for the basic dimensions would be nearer to actual conditions. If cross-sections could be taken simultaneously 10 miles apart over a 100-mile stretch of the Lower Colorado River, weekly between March 15 and the day of the flood peak in late May or June, actual data would be available for interesting comparisons.

If a suggestion has been made pointing to a less cumbersome solution to the problem of soil exclusion at the new diversion than that of constructing the necessarily enormous desilting basins, the effort taken to prepare the paper will have given some satisfaction. A desilting structure on the lines suggested, will be as efficient as the basins; it will be more economical in first cost of construction and in operation; and it will be far less unwieldly because flow velocities are not interrupted. With such a structure available when the river flow has become clear, there will be no reason for the basins, but without it they will always be necessary.

The conclusions are briefly stated as follows:

(1) The volumes of soil transported by the Colorado River have been greatly misconceived, chiefly due to the generally accepted viewpoint that

[32] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1130.

the bed load of flood discharges is stirred up to constitute the suspended load. This is disproved by the greatness of the cubic contents moved by recurring deep scouring during short periods, at least once each year and almost without exception.

(2) The bed load is the principal source of trouble; it contains a much smaller percentage of the fine silts than the suspended load. The difference in the shapes of soil particles offers an explanation for the chief characteristics that distinguish between the bed load and the suspended load, respectively. Granular shapes, when raised into suspension by vertical eddies, tend to sink when they are released from the vertical influence. Therefore, in transportation, they are most dense close to, and along, the bed. On the other hand, the laminar shapes, which in quiescent water sink with their flat sides approximately horizontal, are susceptible to diffusion and retention with the least movement of the fluid in which they are supported, so that flow conditions find the suspended silts uniformly distributed throughout the stream section, for the entire length of the river or canal.

(3) The exclusion of the bed load from any large diversion flow is more important and can be accomplished more easily than that of the suspended load.

(4) Desilting basins of minimum size will trap the bed load by retarding the velocity of flow, so that it can be disposed by subsequent sluicing operations. However, they will not desilt the suspended loads unless it is possible to stop the flow completely for a number of hours. This requirement calls for desilting areas of such magnitude as to be cumbersome and economically non-feasible.

(5) Silty canal flow, without a traveling bed load, has several advantages to the reclamation of 300 000 acres of undeveloped lands on porous soils.

(6) Turbid conditions will gradually lessen and practically cease to exist after an unknown period of several years, when the changed river gradients have become established below the Hoover Reservoir, due to the clear-water discharge from it.

(7) Bed sand will always be in transportation, even when clear-water flow has been established, the silts being first washed from the alluvial channel bed, by meandering of the stream and recession of gradients. With flood velocities modified, the transportation of sand will be slower but continuous, due to additional supplies being contributed yearly from tributary streams and sand washes. This is also due to the fact that the bed becomes depleted of the cohesive silts, and the stream area will then acquire a wide and shallow section, and relatively a greater bed surface for transportation; the velocities, even with minimum discharges, will always be sufficient for moving sand grains.

(8) It is possible to build a diversion structure, with stream approach control, such that it will exclude the traveling bed load automatically, and this would eliminate the necessity for cumbersome sand-trapping basins (the word, "desilting," is not truly applicable) and desludging operations. Such a structure will save money in the first cost of construction and in the costs of operation and maintenance.

(9) Construction of the new head-gate will probably not proceed before the Hoover Dam is advanced sufficiently toward completion to provide effective control of up-stream floods. In the meantime, competent investigation and research on problems pertaining to the silts and sands in transportation, especially within the range of those discharges expected in the future, and on the efficiencies of desilting that can be obtained, will eliminate much of the present guesswork and will enable development of the works to proceed on sound lines, from established findings. Certainly, this is a justifiable aim in view of the permanence and importance of the control structure.

ACKNOWLEDGMENT

Material for the preparation of Figs. 5, 6, 7, and 8 of this paper, was made available through the courtesy of Raymond F. Walter, M. Am. Soc. C. E.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS

GEORGE WASHINGTON BRIDGE: DESIGN OF SUPERSTRUCTURE

By Allston Dana,[1] and Aksel Andersen,[2] Members, Am. Soc. C. E., and George M. Rapp,[3] Assoc. M. Am. Soc. C. E.

Synopsis

This paper gives a general description of the superstructure of the George Washington Bridge and an excerpt of the design specifications which contain certain unusual provisions. It covers in particular a detailed description of the design of the cables and floor system. It also records the weight of the structure as built to date (1932) and of the future additions as planned, and the principal quantities of the materials in the main bridge.

General Description

The main structure of the George Washington Bridge is of the suspension type, with a center span of 3 500 ft. between centers of towers, and suspended side spans of 650 and 610 ft. from center of tower to turning point at the anchorage on the New York and New Jersey sides, respectively. It is designed to carry a 90-ft. roadway flanked by two 10-ft. sidewalks on an upper deck and four electric railway tracks on a lower deck.[4] Only the upper deck has been built to date (1932) and the center portion is unpaved so that the bridge provides two 28-ft. 9-in. roadways and two sidewalks. The clear height under the completed bridge will vary from 195 ft. at the New York pierhead line to 210 ft. at the New Jersey pierhead line. Fig. 1 is a general view of the structure, and Fig. 2 shows the general plan and elevation of the present structure.

Note.—Discussion on this paper will be closed in March, 1933, *Proceedings*.

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[2] Asst. Engr. of Design, Bridge Dept., The Port of New York Authority, New York, N. Y.

[3] Asst. Engr., The Port of New York Authority, New York, N. Y.

[4] "George Washington Bridge: General Conception and Development of Design," by O. H. Ammann, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., August, 1932, Fig. 11, p. 995, and Fig. 20, p. 1010.

The cables, towers, and anchorages are designed for the final loads, which are as follows: Equivalent uniform dead load in center span, 39 000 lb. per ft. of bridge; dead load in side spans, 40 000 lb. per ft. of bridge; and live load in all spans, 8 000 lb. per ft. of bridge.

The assumed temperature variation from a normal of 50° was ± 55 degrees. The wind load was taken as 600 lb. per ft. on each deck and 300 lb. per ft. on the cables.

The cables, which are made up of parallel steel wires, are 3 ft. in diameter, and are arranged in pairs on either side of the roadway, with a distance of 106 ft. between centers of pairs, and a distance of 9 ft. between centers of cables in each pair. They have a center-span sag of 325 ft. under final dead load, and are about 15 ft. above the roadway at mid-span.

The floor structure is hung from the cables at each panel point by means of wire rope suspenders. The floor is cambered with its high point at mid-span, the profile in the center span consisting of two parabolic curves to which the grades of the side spans are tangent at the towers. The New York side span is on a 2.2% grade, while the New Jersey side span is on a 0.4% grade, the roadway being at a higher elevation at the New Jersey end than at the New York end, which takes up in part the difference in the height of land at the two ends of the bridge. The tops of both towers, however, are at the same elevation and the side-span cables slope from the towers at the same angle, but the turning point is set higher at the New Jersey anchorage, which accounts for the difference in side-span lengths.

The cables are supported in saddles on the top of two steel towers at an elevation of 590 ft. above mean high water. Each tower consists of two legs, separated sufficiently to allow the roadway to pass between them, and braced together at the top and just below the floor. Each leg is composed of an inner and an outer row of four columns, one pair of cables being supported over the inner row of columns of each leg. A full description of the design of the towers will be given in another paper of this series.

At the turning points in the anchorage the cables are supported in saddles bearing on concrete buttresses and are deflected downward, the individual strands flaring apart in a distance of 90 ft. to connections with eye-bar anchor chains. These chains are embedded in concrete for a distance of 112 ft. in the New York anchorage and 150 ft. in the New Jersey anchorage and connect to anchor girders at their lower ends.

The New York anchorage is a U-shaped concrete structure consisting of a large block, in which the chains and girders are embedded, connected by low ribs to two forward buttresses supporting the cable saddles. In its completed state the anchorage is designed to have a granite facing that will give it the appearance of a single massive block of masonry, 300 ft. long, 190 ft. wide, and 125 ft. high. The weight of the completed anchorage will be about 520 000 000 lb. The resultant of the maximum cable pull of 247 000 000 lb. and the anchorage weight will have a slope of 1.85 on 1, and the maximum rock pressure will be about 12 tons per sq. ft. On the New Jersey side the anchorage chains and girders are set in rock tunnels filled with concrete.

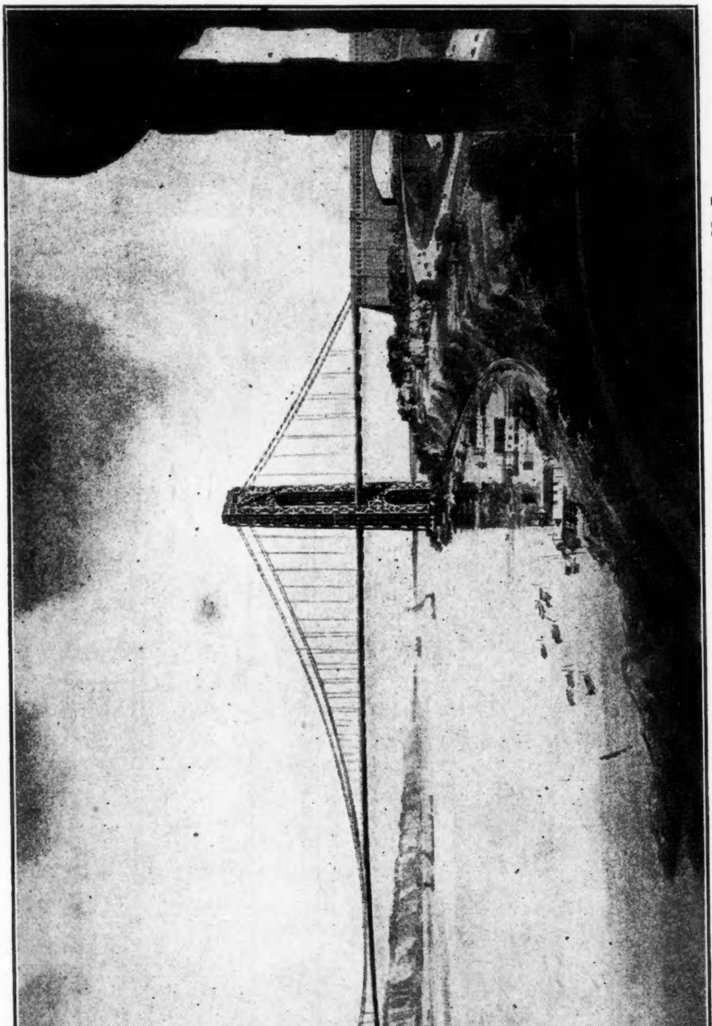


FIG. 1.—VIEW OF GEORGE WASHINGTON BRIDGE, 179TH STREET, NEW YORK, N. Y., TO FORT LEE, N. J.

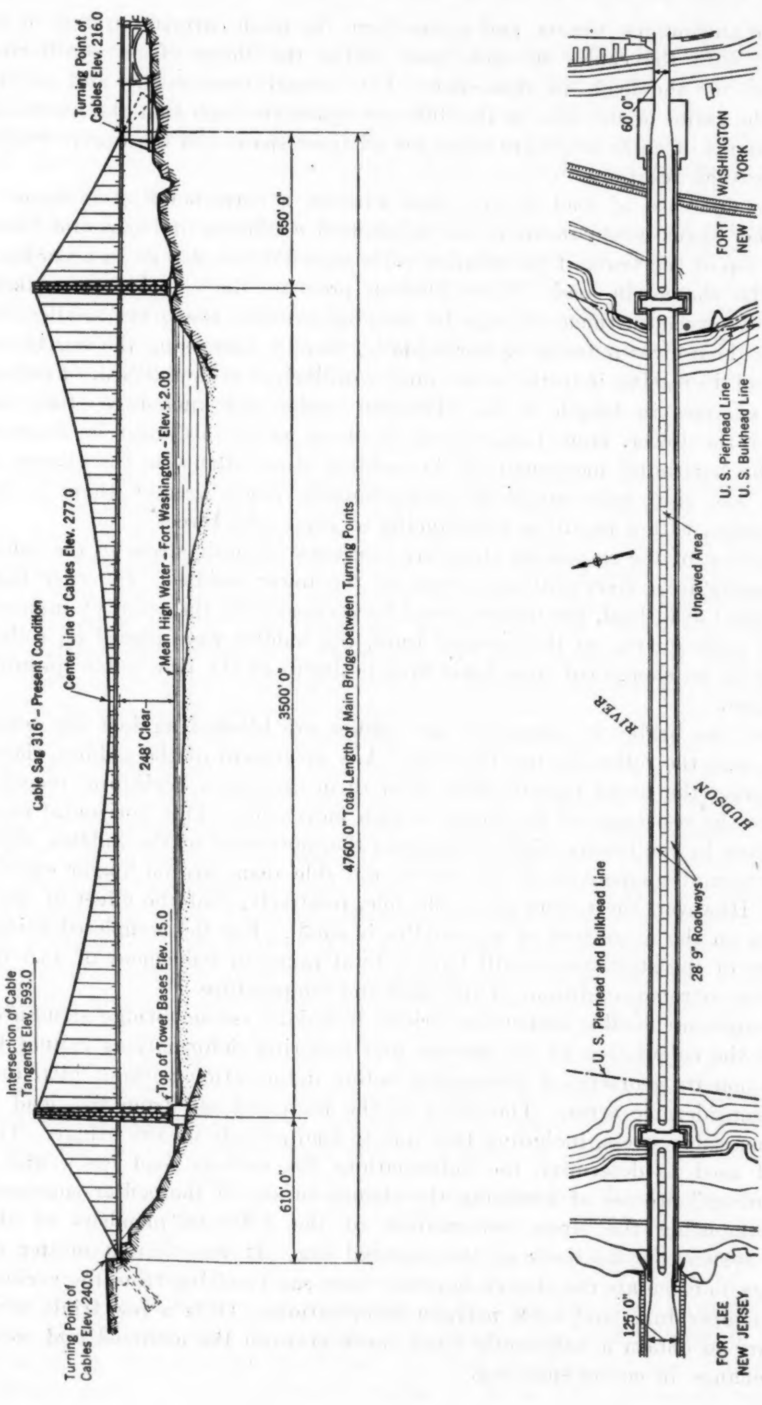


FIG. 2.—GENERAL PLAN AND ELEVATION OF BRIDGE PROPER.

The anchorages, towers, and cables form the main carrying system of the bridge, with the cables in each span taking the shape of an equilibrium polygon for the loads in that span. For normal temperature and no live load, the ratios of the sags in the different spans are such that the horizontal components of cable stress are equal for all three spans and the towers receive only vertical reactions.

Any increase of load in one span without a corresponding increase in the other spans would result in an unbalanced condition of horizontal forces at the top of the tower if the relative cable sags did not change in accordance with the change in load. Since friction prevents the cables from making the necessary adjustment of sags by slipping through the tower saddles, the saddles themselves must move horizontally, thereby increasing the sag in one span and decreasing it in the other, until equilibrium is established. Furthermore, changes in length of the side-span cables and anchorage steel, and even of the tower, from temperature or stress variations must be compensated by horizontal movement of the saddles, since otherwise the change in sag in the side span would be proportionally much greater than in the center span, with a resulting unbalancing of horizontal forces.

Erection of the suspended structure increases the unit stress in the cables and results in a riverward movement of the tower saddles. In order that, under final dead load, the towers would be vertical with the point of intersection of cable curves on their center lines, the saddles were placed on rollers and set 23 in. shoreward from their final positions at the time cable spinning was begun.

With the bridge in operation the saddles are blocked against the tower tops so that the rollers do not function. Any movement of the saddles, therefore, forces the tower tops to move with them causing a horizontal reaction equal to the resistance of the tower to such movement. This horizontal reaction taken by the towers slightly dampens the movement of the saddles, since the horizontal components of the center and side spans are no longer exactly equal. However, the towers are so flexible, relatively, that the effect of their stiffness on the movement of the saddles is small. For the completed bridge the tops of the steel towers will have a total range of movement of 13.3 in. under the extreme conditions of live load and temperature.

A long-span flexible suspension bridge is unique among bridge structures in that the calculation of its stresses and resulting deformations cannot be based upon its geometrical dimensions before deformation without introducing a considerable error. The effect of the increased sag from live load is to reduce cable stress, including that due to load already on the bridge. The method used to determine the deformations for various load cases was a "cut-and-try" process of assuming the change in sag of the center span and then computing the stress deformation of the different members of the elastic system on the basis of the changed sag. It was then a matter of geometry to calculate the change in center-span sag resulting from the various stress, temperature, and cable polygon deformations. Only a few trials were necessary to obtain a sufficiently close check between the assumed and computed change in center-span sag.

At present, with only the upper deck in place, the bridge has no stiffening trusses. The dead load resists the live load deformation of the cable polygon, and, because of its great magnitude, offers ample stiffening effect. When the lower deck is built the bridge will have two stiffening trusses, 29 ft. deep and 106 ft. apart. The upper chords of the stiffening trusses are now in place and form the chords of the wind trusses.

Subsequently in this paper detailed descriptions are given of the design of the cables and the floor system as built. Some of the features described were incorporated in the alternate designs shown on the contract drawings on which bids were taken, while others were developed after the contract was awarded and the type of cables and method of erection were determined. No attempt is made in these descriptions to designate at what stage in the design any particular detail was established.

Design Specifications

Most specifications for the design of bridges in general, are scarcely applicable to long-span bridges. In fact, practically every long-span bridge has been designed in accordance with a special set of specifications intended to apply to that bridge alone. It was considered desirable to have a set of specifications, flexible and broad enough to be used for all bridges that the Port Authority was then planning and might design in the future. Accordingly, such specifications were drawn up, and have been applied, with slight modifications, to slabs, beams, and arches of concrete, as well as to long-span cantilever, arch, and suspension bridges.

Live Load.—A novel feature of the Port Authority specifications is the treatment of the live load. It was realized that the intensity of live load, for which any part of a bridge structure should be designed, decreases as the extent of the area required to be loaded for maximum stress increases. This is true, partly because the probable average congestion and percentage of heavier vehicles decreases as the loaded area increases; and also because those parts of the structure, which require a large loaded area for maximum stress, have a higher percentage of dead load stress and are, therefore, better able to carry a live overload. Therefore, a uniform live load is specified which has a variable intensity, depending upon the loaded length and number of loaded lanes.

The basic uniform loads are: 100 lb. per sq. ft. of sidewalks, 250 lb. per sq. ft. of roadways, and 6 000 lb. per ft. of electric railway tracks. These basic loads are to be multiplied by two factors, K and C, which are functions, respectively, of the loaded length and the number of loaded lanes. Factor K is equal to $0.2 + \dfrac{160}{200 + l}$, in which, l is the loaded length, in feet; and C is equal to $0.5 + \dfrac{2}{n + 3}$, in which, n is the number of lanes loaded, considering a 10-ft. width of roadway as one lane. Where the live load is made up of lanes of different load intensities, the number of loaded lanes is to be obtained by dividing the total load per foot by the load per foot of the heaviest loaded lane; a minimum value of K of 0.25 corresponding to a

loaded length of 3 000 ft., and a minimum value of C of 0.682 corresponding to eight lanes loaded, are specified. Thus, the minimum value of $K \times C$ is 0.170.

The uniform load is to be used alone, and not in combination with concentrated loads. However, since it is scarcely feasible to use uniform loads in the design of members receiving their maximum stress from very short loads, concentrated loads are also specified. These loads are to be used as alternatives to the uniform loads, depending on which gives the larger stress. Fig. 3 shows the axle concentrations. Not more than one truck on any one lane, or the four axles on any one track, are to be considered. However,

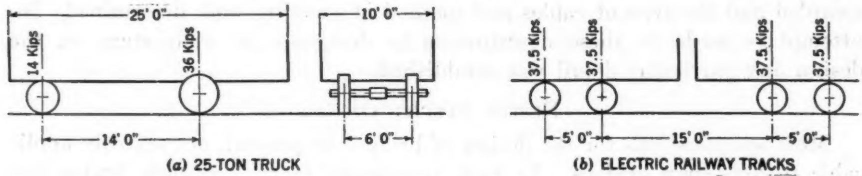


FIG. 3.—AXLE LOADS.

any number of lanes may be loaded, providing the concentrations are reduced by multiplying by the factor, C, which is the same as for uniform loads.

The ratio of impact to live load is, $\frac{150}{200 + l} \times \frac{4}{3 + n}$, in which, n and l are the same as in the live load formulas; l is to be taken as zero for concentrated loads. No impact is used for loaded lengths of more than 1 500 ft.

Applying the specifications to the main carrying system of the George Washington Bridge, and using the width of roadway and sidewalks assumed at the time the design was made, the following live load, unreduced for loaded length and number of lanes, is obtained:

80-ft. roadway @	250 lb. per sq. ft.	= 20 000 lb. per ft.
Four electric railway tracks @	6 000 lb. per ft.	= 24 000 lb. per ft.
Two 10-ft. sidewalks @	100 lb. per sq. ft.	= 2 000 lb. per ft.
Total		= 46 000 lb. per ft.

As the loaded length is more than 3 000 ft. and the number of loaded lanes is eight, this load was multiplied by the minimum values of K and C, giving $46\,000 \times 0.170 = 7\,820$ lb. per ft., which was rounded up to 8 000 lb. per ft.

For maximum bending of the towers, certain spans should be unloaded. It was considered too severe an assumption to place maximum live load on some spans and no live load on other spans. It was assumed, therefore, that no span would be loaded with less than one-half the design live load.

Most of the stiffening truss members receive their maximum stress from a comparatively short load. Since the stiffening truss is not a part of the main carrying system of the George Washington Bridge, an unreduced partial live load of 23 000 lb. per ft. of bridge, or one-half that given for the main carrying system, to be reduced by the factors, K and C, was specified and

also an extended load of 4 000 lb. per ft., or one-half the reduced load given for the main carrying system placed over the entire bridge.

Wind Load.—The following wind pressures are specified to act as continuous, uniformly distributed loads of any length or position to produce maximum stress: A wind pressure of 300 lb. per lin. ft. along each deck, acting 6 ft. above the top of rail or roadway surface and at any track or lane of roadway, and a wind pressure of 30 lb. per sq. ft. of area of exposed structure. The leeward truss is considered fully exposed except where covered by solid floor structures or trains and vehicles. The wind loads used in the design of the George Washington Bridge of 600 lb. per ft. on each deck and 300 lb. per ft. on the cables are round figures arrived at from the specifications. The resulting wind loads are assumed to act either at right angles to the axis of the bridge, or at an angle of 45°, with transverse and longitudinal components each equal to two-thirds of these wind loads.

Lateral Force.—The specified lateral force for bridges carrying electric railway tracks is 10% of the live load on any two tracks, assumed to act 6 ft. above the top of rail.

Longitudinal Force.—The specified longitudinal force from breaking or traction is 20% of one 25-ton truck on each of any two lanes of the roadway and 20% of the specified uniform live load, not exceeding 1 000 ft. in length, on any two tracks. The forces from the two trucks or trains may act in the same or in opposite directions. This longitudinal force was found to be 720 000 lb. for the center span of the George Washington Bridge.

Permissible Unit Stresses.—The following tabulation gives the permissible unit stresses, in pounds per square inch:

Tension, Net Section:

Rolled carbon steel	20 000	
Rolled silicon steel	27 000	
Rolled nickel steel	33 000	
Heat-treated eye-bars, low strength	33 000	For total stress, inclusive of secondary stress
Heat-treated eye-bars, high strength	50 000	For total stress, inclusive of secondary stress
Reinforcing steel	18 000	For total stress, exclusive of secondary stress
Cold-drawn cable wire.....	82 000	For total stress, exclusive of secondary stress

Compression, Gross Section (in which, l = unsupported length of member, and r = least radius of gyration):

Rolled carbon steel .... $20\,000 - 60\frac{l}{r}$, maximum, 17 000

Rolled silicon steel .... $27\,000 - 80\frac{l}{r}$, maximum, 23 000

Rolled nickel steel .... $33\,000 - 100\frac{l}{r}$, maximum, 28 000

Steel castings 20 000
Iron castings 12 000

Bending on Extreme Fibers:

Riveted Girders and Rolled Beams:

Tension, Net Section:

Carbon steel	20 000
Silicon steel	27 000
Nickel steel	33 000

Compression, Gross Section (in which, l = unsupported length of flange, and b = width of flange):

Carbon steel	$27\,000 - 200\frac{l}{b}$, maximum, 17 000
Silicon steel	$27\,000 - 270\frac{l}{b}$, maximum, 23 000
Nickel steel	$33\,000 - 330\frac{l}{b}$, maximum, 28 000

Forged carbon steel	18 000
Steel castings	15 000
Pins, carbon steel	30 000
Pins, heat-treated	50 000

Shear:

Girder Webs, Gross Section:

Carbon steel	12 500
Silicon steel	17 000
Nickel steel	20 000
Carbon-steel pins	15 000
Heat-treated pins	25 000
Power-driven rivets and turned bolts	15 000
Hand-driven rivets and unfinished bolts	10 000
Steel castings	12 000
Iron castings	5 000

Bearing:

Carbon steel, cast steel, and carbon-steel pins	30 000
Silicon steel	40 000
Nickel steel	50 000
Heat-treated pins	55 000
Power-driven rivets and turned bolts	30 000
Hand-driven rivets and unfinished bolts	20 000
Rollers, per linear inch, in which, d = diameter of roller, in inches:	
Ordinary carbon steel	$800d$
Steel, minimum elastic limit, 60 000	$1\,200d$

All the permissible unit stresses given in the foregoing tabulation are taken from the general Port Authority specifications and were used for the George Washington Bridge with the exception that the following values were used for power-driven rivets and turned bolts: Shear, 12 500 lb. per sq. in.; and bearing, 25 000 lb. per sq. in.

The following combinations of stresses are specified for the proportioning of any member:

1.—Dead load, live load impact, and temperature;
2.—Dead load combined with wind, or longitudinal force, or lateral force.

3.—Lateral force combined with wind, or longitudinal force;
4.—Longitudinal force combined with wind;
5.—Dead load, live load, impact, and temperature combined with either wind load of one-half the intensity of that specified under "Wind Load," or longitudinal force, or lateral force.
6.—Dead load, one-half live load, one-half impact, and temperature combined with wind.

For Combinations 1, 2, 3, and 4, the permissible basic unit stresses as given are to be used. For Combinations 5 and 6, unit stresses 10% higher than the basic unit stresses may be used. The permissible unit stresses for the combination of primary stresses and secondary stresses are 25% greater than those specified for primary stresses alone.

It will be noted that the basic permissible unit stresses are roughly 60% of the yield-point stresses, and that the permissible stresses for extreme combinations of primary stresses and secondary stresses are roughly 83% of the yield-point stresses.

Design of Cables, Anchorages, and Suspenders

Cables.—Each cable is composed of 26 474 galvanized steel wires, 0.196 in. in diameter (No. 6 B. W. G. (Birmingham wire gauge)), over galvanizing, laid parallel to form a compact cylinder 36 in. in diameter. There are 800 sq. in. of steel area in each cable, and the length along the cable from strand shoes to strand shoes is nearly 1 mile. The total weight of wire in the four cables is 56 600 000 lb., and the total length of single wire is about 105 000 miles, or enough to extend slightly more than four times around the earth at the equator.

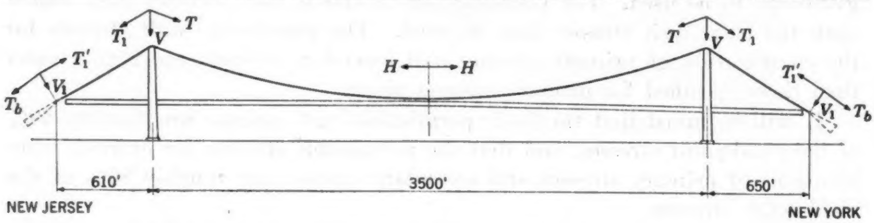
The cable wire is cold-drawn steel having a specified minimum ultimate strength of 220 000 lb. per sq. in., and a minimum yield point of 150 000 lb. per sq. in. The actual wire furnished by the contractor showed somewhat higher strength, however, the ultimate strength averaging about 234 000 lb. per sq. in., and the yield point about 184 000 lb. per sq. in.

The cables are designed for a maximum unit stress of 82 000 lb. per sq. in., exclusive of secondary stresses. This maximum stress occurs in the side spans adjacent to the tower saddles. The maximum direct unit stresses in the cable at other points are in direct proportion to the secant of the angle of inclination of the cable with the horizontal: In the center span adjacent to the tower, it is 74 000 lb. per sq. in., or 10% less than the maximum; at the middle of the main span, it has a minimum value of 69 000 lb. per sq. in., or 16% less than the maximum; at the anchorage saddles and in the backstays adjacent to the eye-bars, it is 78 000 lb. per sq. in., or 5% less than the maximum. Table 1 shows the forces in the cables at the towers and anchorages.

On account of the relatively short, stiff side spans and the resulting small angular distortions of the cable, the maximum secondary stresses do not occur at the point of maximum direct stress, however, but rather on the opposite side of the saddle in the center span. To determine the intensity of the secondary stresses in the cable at this point field measurements of strain were

made on 150 wires on the periphery of the cable during different stages of construction when the cable distortions were large and varying. (See Table 2.) A study of these measurements and the measurements of angular deflection that were made simultaneously with them—when applied to the maximum direct stress and simultaneous angular deflection of the cable on each side

TABLE 1.—FORCES, IN MILLIONS OF POUNDS, FOR FOUR CABLES



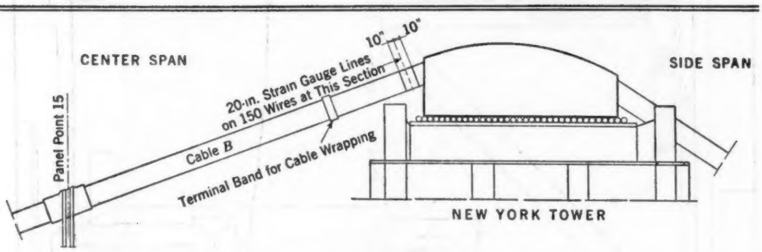
	T_b	V_1	T_1'	T_1	V	T	H	T	V	T_1	T_1'	V_1	T_b
Dead	208	34	208	218	184	196	184	196	185	218	207	34	207
Live	38	7	38	40	36	36	34	36	36	41	39	7	39
Temp.	2	0	2	2	1	2	2	2	1	2	2	0	2
Total	248	41	248	260	221	234	220	234	222	261	248	41	248

of the tower saddle—indicates that the bending stresses at these points will be approximately 2 000 lb. per sq. in. on the side-span side and 5 000 lb. per sq. in. on the center-span side.

The determination of the sectional area of the cables is comparatively a simple matter. They must carry, in addition to their own dead weight, the dead weight of the entire suspended structure, and practically all the superimposed live load. Strictly speaking, the cables are statically indeterminate to the first degree because they form only one member of a two-member carrying system, the other member being the stiffening trusses. It must be evident, however, that a truss so shallow in comparison with its length as that planned for the center span of the George Washington Bridge, can carry very little load by beam action, and, therefore, that practically all the live load in the center span is carried by the cables. The side-span trusses, being shorter and, therefore, much stiffer, will carry an appreciable amount of live load directly to their supports and thus relieve the cables; but this effect is almost negligible. The problem of determining the maximum cable stress, therefore, reduces to taking moments of the total dead and live load about the low point in the cable (mid-span), dividing by the sag of the cables, and multiplying by the secant of the maximum inclination of the cables, with the horizontal. However, as the live load is added the cables stretch and sag deeper so that they carry the entire load on a larger sag than before the imposition of the live load. In other words, the lever arms of the forces change by appreciable amounts during application of the load, and this effect, negligible in the design of most structures, must be considered. By a process of "cut-and-try," or by means of a more complicated direct solution, the maximum tension in the cable is found. This, divided by the unit stress allowed, gives the required sectional area of steel.

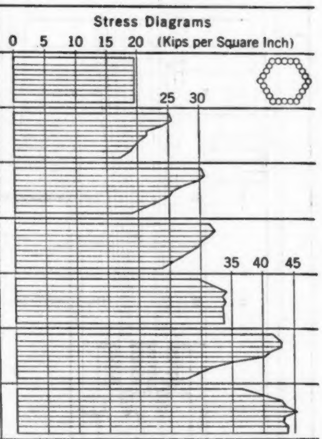
In proportioning the cable certain factors had to receive consideration: For convenience in erection and in anchoring, the cable is divided into component parts called strands, each strand having the same number of wires. The number of wires must be limited so that the weight of the strands is not too great for easy handling in the field during erection. It was decided

TABLE 2.—Results of Cable-Bending Stress Measurements



(*a*) LOCATION OF STRAIN MEASUREMENTS ON CABLE FOR DETERMINATION OF EFFECT OF BENDING

Erection Stage	Deflection at PP15 in Ft.		Average Stress in Kips per Square Inch	
	Measured	Calculated	Measured	Calculated
N. J. PP43 N. Y. (*b*)	0	0	Starting Point	19.6
(*c*) 18 18	1.54	1.59	21 5	20.8
(*d*) 23 22	1.97	1.89	25.6	22.7
(*e*) 32 32	1.65	1.41	28.6	27.2
(*f*) All Steel Erected	0.37	0.38	32.8	32.9
(*g*) Concrete Mixer 31	1.49	1.52	36.5	35.2
(*h*) Bridge Completed	0.70	0.74	41.9	41.6



that 61 strands of 434 wires, $4\frac{1}{2}$ in. in diameter—requiring lifts at the tower of about 110 tons and pulls at the anchorages of about 130 tons—would not impose unreasonable requirements in the design of the erection equipment, and would give fewer units to handle compared with the next alternative of using 91 lighter strands. As considerable erection time is consumed in handling and adjusting the completed strands there is an appreciable saving resulting from the use of the 61-strand assembly. The use of 61 rather than 91 strands also required fewer anchorage eye-bars and strand shoes, which resulted in a simpler and more economical anchorage.

The 61 strands are laid up in the form of a hexagon during the construction of the cables and, later, are compacted to a circular shape throughout the three spans. Where they pass through the saddles on top of the towers

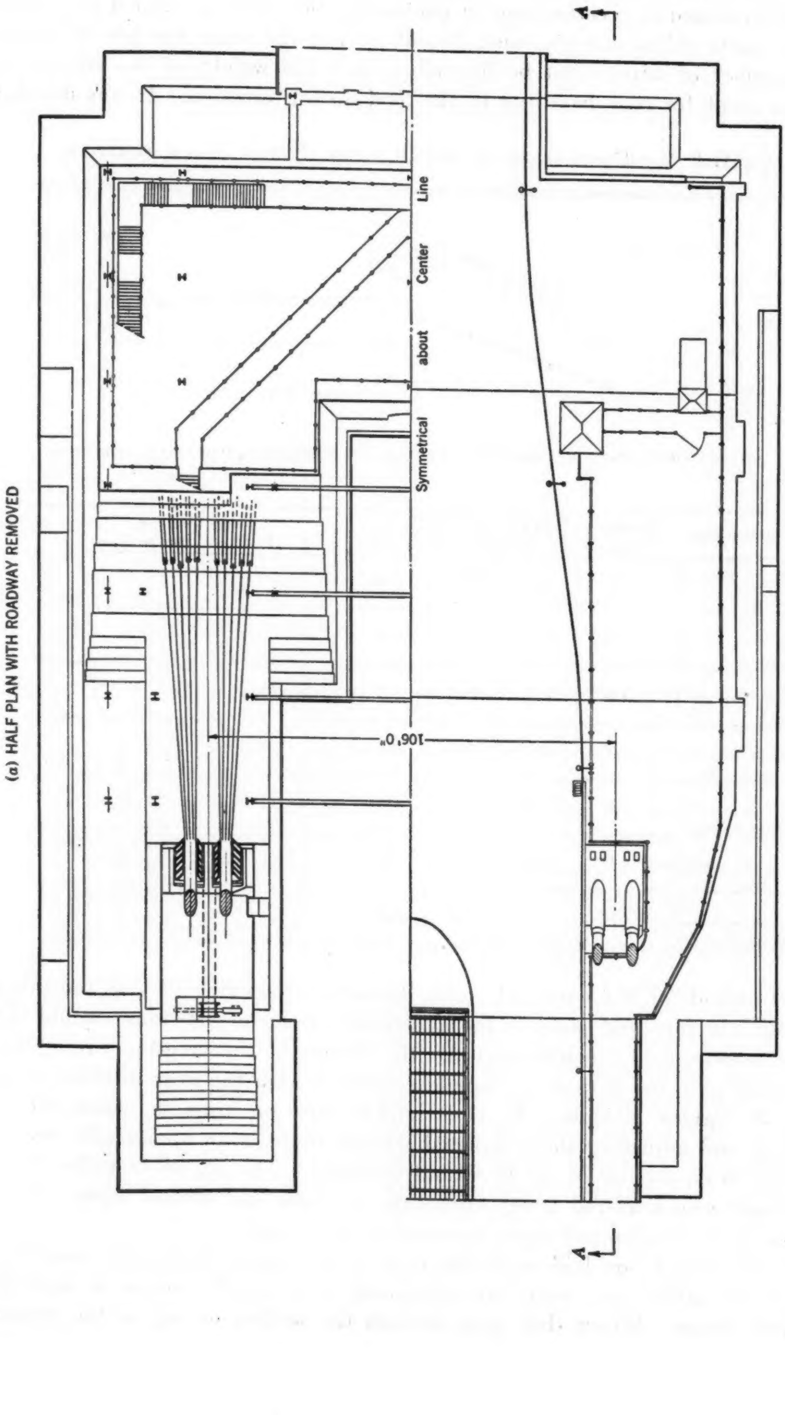
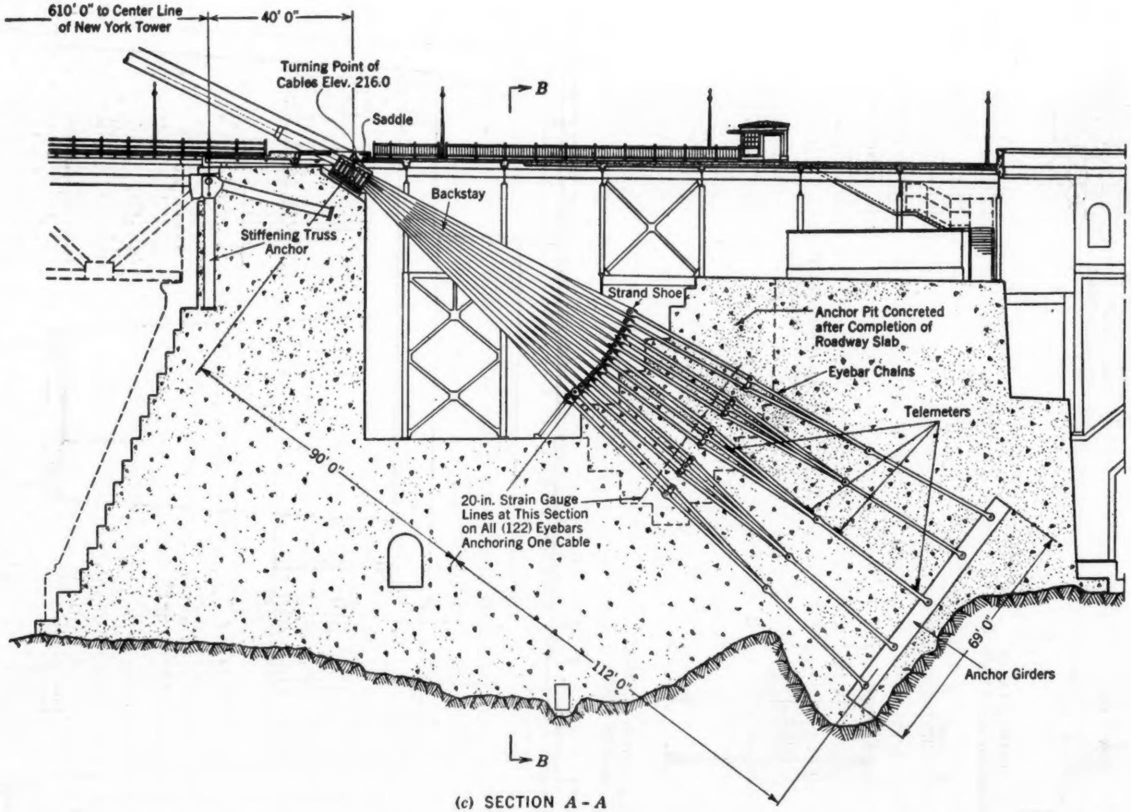


FIG. 4.—PLAN OF NEW YORK ANCHORAGE (SEE, ALSO, FIGS. 5 AND 6).



(c) SECTION A - A

FIG. 5.—LONGITUDINAL SECTION A-A, NEW YORK ANCHORAGE (SEE, ALSO, FIGS. 4 AND 6).

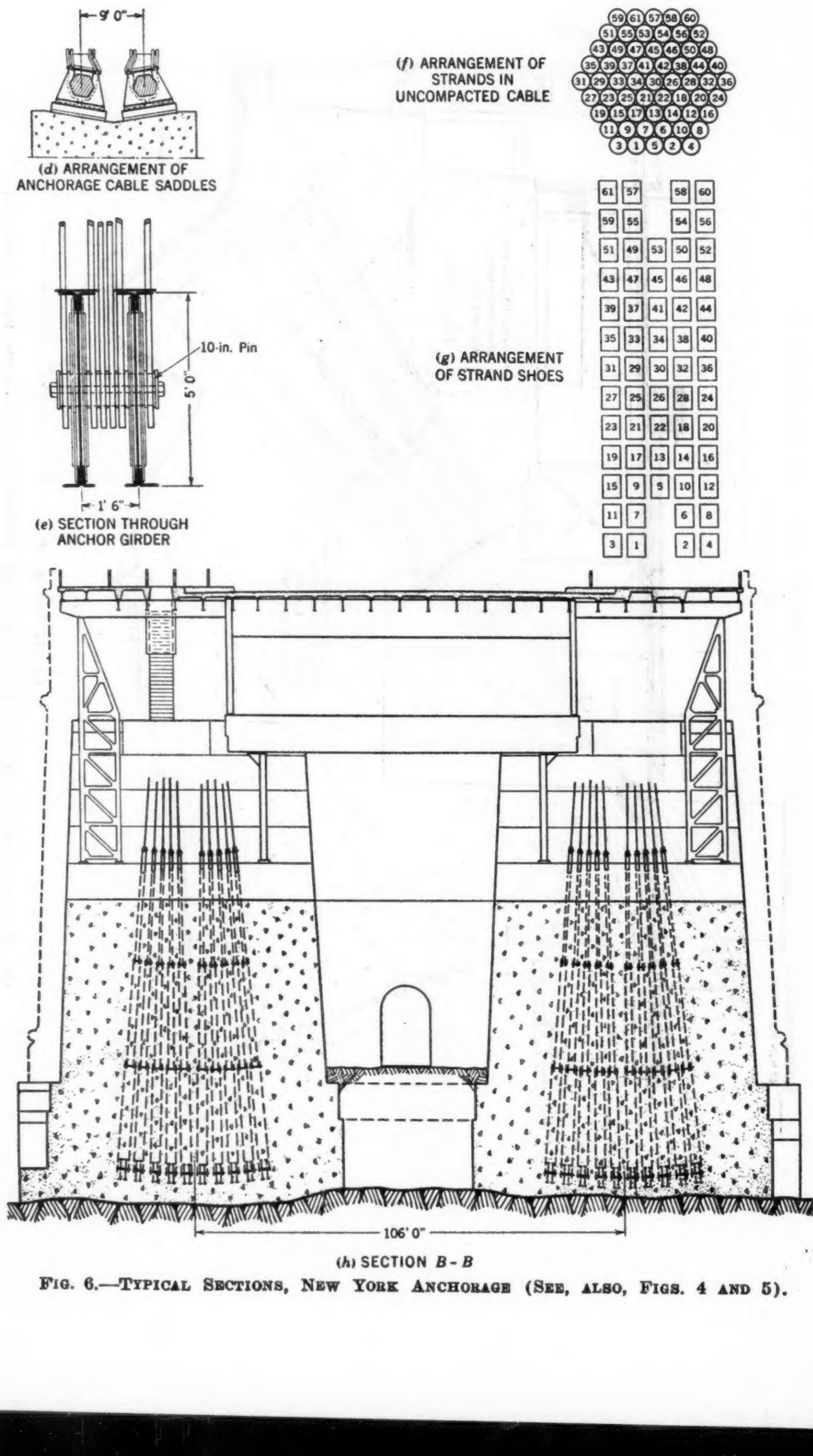


FIG. 6.—TYPICAL SECTIONS, NEW YORK ANCHORAGE (SEE, ALSO, FIGS. 4 AND 5).

and at the anchorages, however, the cables retain their original hexagonal shape, the saddles being designed for that condition.

In order to design the cable bands that support the suspenders, some knowledge had to be obtained beforehand of the approximate diameter of the compacted cables. This was determined by applying the formula, $D = k \times 1.05 \times d \sqrt{N}$, in which, D is the diameter of the cable, k is a constant, d is the diameter of the individual wires, and N is the number of wires. The constant, k, was determined from a study of the diameter of the cables of several other large suspension bridges. Expressed in another way, there is a minimum of about 10% voids in a perfectly compacted group of wires, but as this ideal condition cannot be obtained, there is found by experience to be about 10 to 12% additional voids. On this basis, it was calculated that the diameter of each cable would be 36 in. Later, as a result of experimental compacting on a full-sized section of sample cable, constructed to determine the bore of the cable bands, it was found that the cable could be squeezed to a diameter of $35\frac{7}{8}$ in., which gives 21% of voids.

Anchorage Steelwork.—The maximum cable stress, cable section, and number of strands being determined, one of the next problems was to design the steel anchorages. The positions of the turning points or anchorage-cable saddles were determined so as to be below the main floor deck, and to give approximately the same slope to the cables in the two side spans. From these points the cables deflect downward to the anchorage steel through a vertical angle of 9° 28′. The two cables of each pair separate, each cable deflecting through lateral angles of 1° 34′ in New York and 1° 19′ in New Jersey, and the 61 strands of each cable flare or splay apart both vertically and laterally, each strand connecting at its end to an independent pair of eye-bars. (See Figs. 4 to 9.)

The vertical deflection angle of the cables was made the same at both anchorages in order to keep the details of the saddles the same; it was made sufficiently large, but no larger than necessary, to prevent the topmost strands from lifting off the saddle in the worse condition of upward deflection in the side span. The deflection angle of the top strands, therefore, is only about 1 degree.

The lateral deflection of the cables was necessary because the 9-ft. spacing of the cables (which was made the minimum consistent with the space required for spinning, compacting, and wrapping the cables, and for the design of the saddles), could not be maintained in the anchorages on account of the greater space required for the anchor steel. The cables, therefore, diverge in two straight lines from the 9-ft. spacing at the saddles to a 20-ft. spacing at the bottom of the anchorage steelwork, where the ten double girders are placed with the minimum lateral spacing of 4 ft., center to center. (See Figs. 6 and 9.)

In order to avoid the use of tension ties at the turning point between the cables to take the lateral thrust caused by these horizontal deflections, the anchorage saddles were rotated about the center line of the side-span cables at that point until they were in the oblique plane formed by

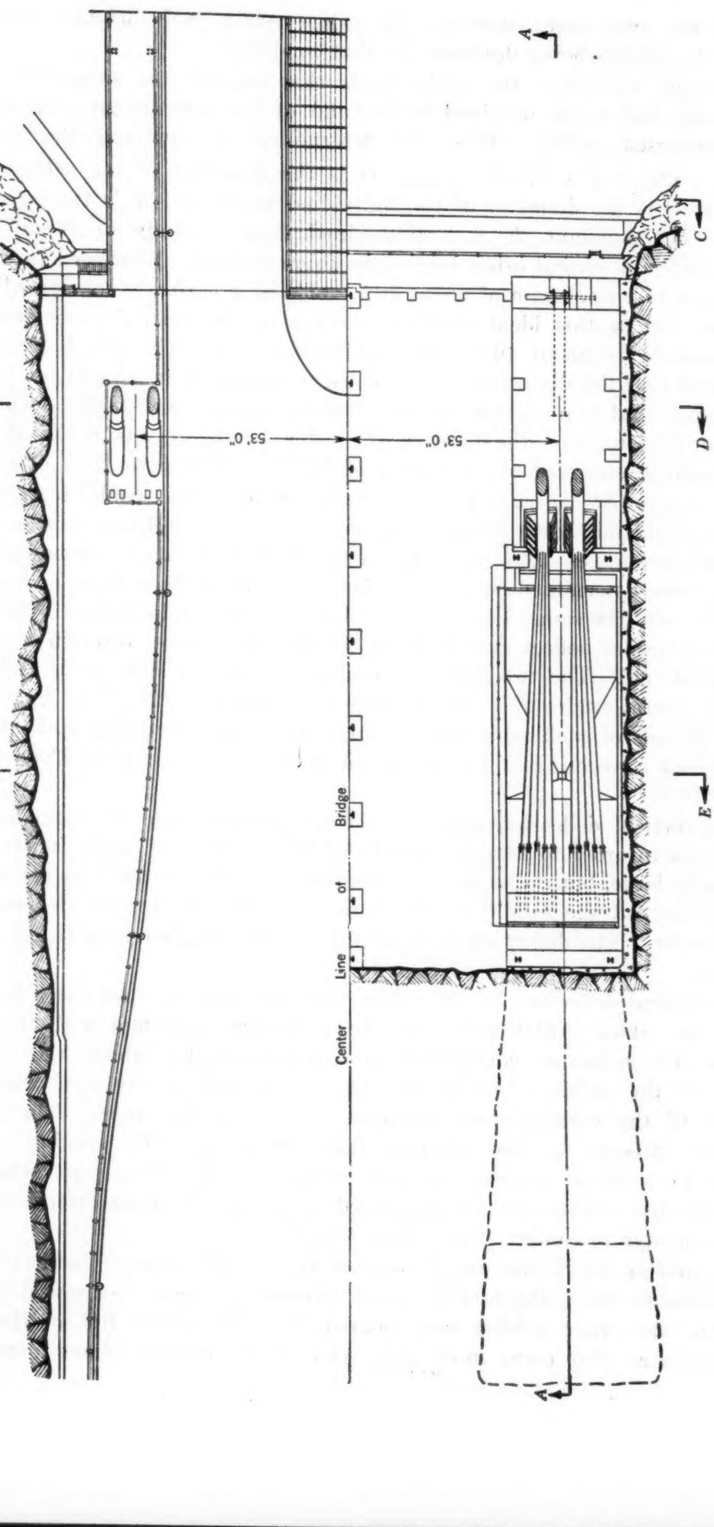


FIG. 7.—PLAN OF NEW JERSEY ANCHORAGE (SEE, ALSO, FIGS. 8 AND 9).

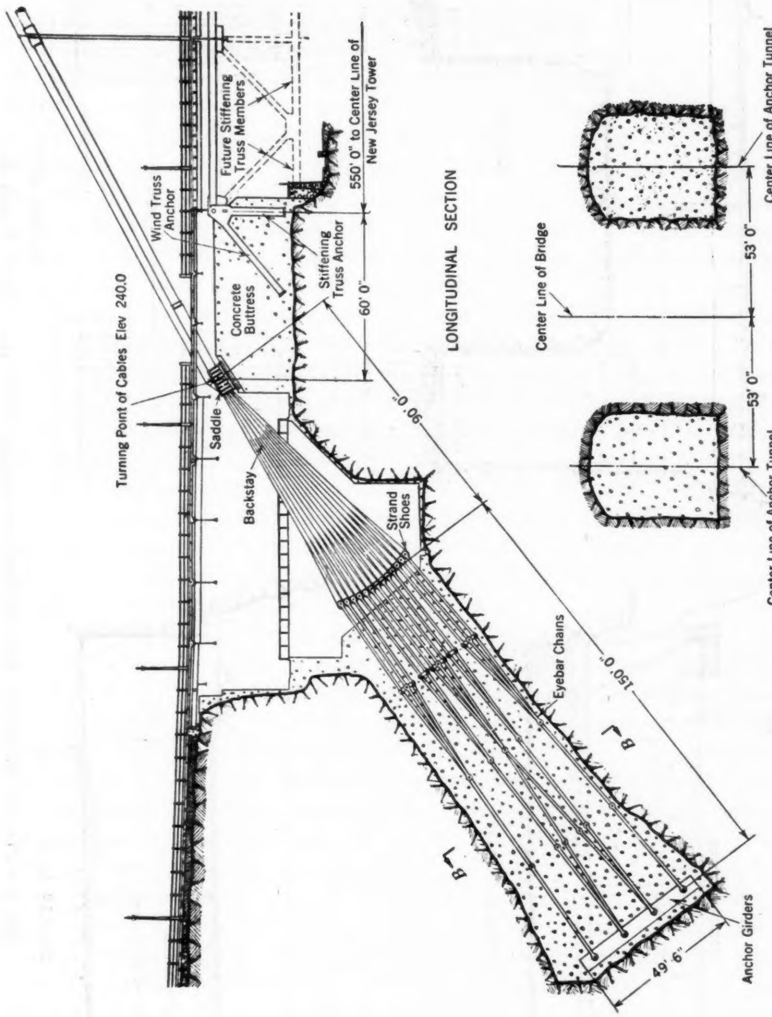


FIG. 8.—LONGITUDINAL SECTION *A-A*, NEW JERSEY ANCHORAGE (SEE, ALSO, FIGS. 7 AND 9).

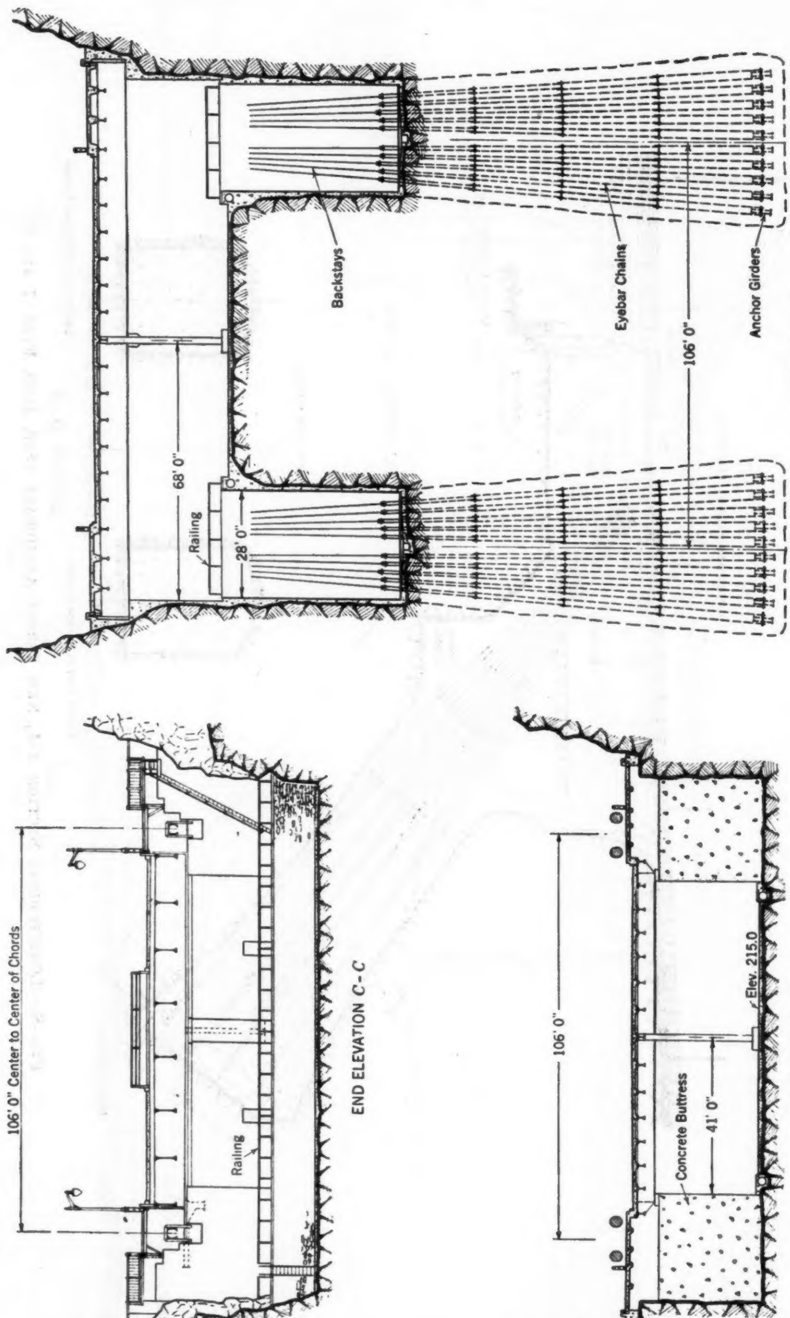


FIG. 9.—TYPICAL SECTIONS, NEW JERSEY ANCHORAGE (SEE, ALSO, FIGS. 7 AND 8).

this line and the center line of the splayed back-stay cable and anchorage steel. This placed them in planes making dihedral angles with the vertical planes of the side span cables equal to 9° 24′ and 7° 56′, respectively, on the New York and New Jersey sides. In this way the resultant of the pulls in the side span and back-stay cables is a simple thrust against the masonry of the concrete buttresses.

Instead of supporting the cables in their hexagonal shape in the conventional grooved saddles, thence passing to a circular shape again and into separate splay collars from which the strands diverge to their anchors, the cables were splayed directly from the mouth of the saddles, and the saddle details were made to accommodate this. This solution was possible because the cables at this point are beneath and are protected by the floor and, therefore, did not require compacting and wrapping for protection against the weather. By eliminating separate splay collars the length of the cables was reduced and their construction simplified.

In the New York anchorage there are three tiers or lengths of eye-bars between the strand shoes and the anchor girders, whereas in the New Jersey anchorage there are four tiers, the distance between the shoes and the girders being 112 ft. in New York and 150 ft. in New Jersey. This greater length of the New Jersey anchors resulted from using a more conservative value for the shearing stress in rock than in concrete. Each vertical row of strand shoes is connected by a chain of eye-bars to a separate anchor girder, the thirteen lines of eye-bars in the upper tier of a typical chain converging in groups of two and three and connecting to the five lines of eye-bars of the lower tiers. Each anchor girder is composed of two plate girders fastened together with diaphragms and batten-plates.

In the New York anchorage the lateral spacing of the strand shoes and that of the girders are in proportion to their distances from the saddle, so that the lateral flare of each eye-bar chain is a continuation of the lateral flare of the strands which it anchors; and the pins in the girders were located so that each line of eye-bars in a chain is a continuation of the group of strands connecting to it. This straight-line pull arrangement made it unnecessary to use lateral separators between the strand shoes, although vertical separators were necessary on account of the converging of the groups of upper tier eye-bars previously mentioned.

A similar straight-line pull arrangement was not practicable in the New Jersey anchorage. On account of the greater length of the New Jersey anchor chains such an arrangement would have required longer girders and a wider spacing between them, whereas it was desirable to keep the size of the rock tunnels, in which the anchorage steel was embedded, to a minimum. The lateral spacing, therefore, was made the same as in the New York anchorage and the girders were made considerably shorter, thus causing a change in direction at the shoes between the strands and the eye-bars both vertically and laterally. This required lateral as well as vertical separators between the strand shoes to take care of the thrusts. Furthermore, during the spinning of the cables, prior to the completion of the top strands, it required special holding down devices to resist the unbalanced upward thrust.

The maximum pull in each cable at the anchorages is 62 000 000 lb. This is assumed to be distributed equally among the 61 strands and pairs of eye-bars. Heat-treated eye-bars were used with a specified minimum yield point and ultimate strength of 50 000 and 75 000 lb. per sq. in., respectively, and with an allowable unit stress of 30 000 lb. per sq. in., exclusive of secondary stress. A sectional area of 16.9 sq. in. was required for each bar, and in all but the upper tier 10 by $1\frac{3}{4}$-in. bars were used. In the upper tier, 10 by $1\frac{7}{8}$-in. bars were used, the heavier section being provided to take care of the small but unavoidable bending stresses to which these bars are subjected. As far as possible all bars in both anchorages were made 38 ft. 4 in. long, which is the maximum for full-sized tests.

The use of ordinary carbon-steel eye-bars instead of the heat-treated bars was considered, but the greater weight and increased packing dimensions would have more than offset the lower unit price.

The strand shoes and girders were set laterally on circular arcs corresponding to the lateral flare of the strands and eye-bar chains. The strand shoes, however, were set vertically on a sharper circular arc centering at a point midway between the shoes and the saddle, thus making the lengths of the flared strands inversely proportional to the secant of their angles of flare and in this way equalizing their stress.

Further precautions were taken to insure an equal distribution of stress in the back-stay strands by eliminating the sag due to the relatively heavy concentrations from the strand shoes, eye-bar heads, and pin details. During cable construction the top tier of bars cannot be concreted in, and the entire assembly, if unsupported, would sag at the expense of an increase of stress in the lower strands. Therefore, column supports were provided under each chain at the strand shoes, which held the strands in their correct final position during and after construction of the cables, and until the eye-bars were embedded in concrete.

Another source of inequality of stress in the back-stay strands is inherent in the successive adjustment of the strands from the bottom up as the cable is built. As each new strand is adjusted for side-span sag it is lowered into place in the anchorage saddle and, simultaneously, the shoe is pulled back to its final position. During the spinning of the strand the shoe is held in a position forward of its final position by a strand leg which is pinned to the anchorage eye-bars. The pulling back is done by a pulling jack and rope which connect to the strand leg and to the pin at the lower end of the upper tier of eye-bars, these bars being released from stress during the process. In order to have clearance for inserting the shims required for the adjustment, the shoe is pulled back a little farther than its final position and as the pulling jack is released the shoe moves forward the amount of this shim clearance and also the amount of the elastic stretch of the upper tier of eye-bars to which the pull is transferred from the jack. This slight forward motion of the shoe allows the strand to slip forward in the saddle, but only to a partial extent on account of the friction of the strand on the strands below in the saddle. Consequently, the stress in the strand is reduced

by the amount of this friction, and the stress in the previously adjusted strands is increased.

This unbalancing of stress in the strands, and its amount, was discovered by strain measurements which were made on all the upper-tier eye-bars anchoring one of the cables during its erection. These measurements were planned at the outset to check the distribution of the cable pull between the strands. Specially designed, manually-operated, 20-in. strain-gauges with a lever ratio of 10 to 1 and 0.0001-in. Federal dials were used, enabling stresses as low as 150 lb. per sq. in. to be observed. Gauge holes were drilled in the eye-bars on the top and bottom faces, directly on the middle line of the bars, in order to eliminate the effects of bending. Strain measurements were taken after every set of four strands had been adjusted into place, all bars then in place being measured each time. Measurements were also taken after the floor steel was erected and again after the concrete roadway slabs were placed. In this way nearly 2 500 strain measurements were made. Table 3 gives the percentage by which the stress in each strand was greater or less than the average measured in the strands for each set of measurements. A study of these results shows that each new set of four strands, as it was adjusted into place, unloaded some of its stress into the strands previously placed, the four lowest strands being stressed to as much as 40% more than the average by the time the cable was completed. The addition of the floor steel and the concrete slabs, as was to be expected, reduced the percentage variation in strand stress. Except in a few cases, the average measured stress agreed well with the calculated average for the stages of construction at which the measurements were made, this agreement being particularly close in the case of the last two sets. In the last set of measurements, which represents practically the present condition of loading on the bridge, the lowest strands had 20% more stress than the average. This is equivalent to a unit stress of 3 000 lb. per sq. in. in the eye-bars, or 10% of the allowable unit stress.

In addition to the strain measurements on the eye-bars adjacent to the strand shoes, measurements were also made, during various erection stages of the bridge, of the strain in some of the eye-bars embedded in the concrete. In the design of the anchorage steel it was assumed that full stress remained in the bars down to the anchorage girders, and all pin details and girder details were based on this assumption. It was realized that undoubtedly much of the stress would be removed from the eye-bars before it reached the girders, but in the absence of some definite knowledge of how much and how fast this relief of stress took place, it was not considered wise to take advantage of it in the design.

It was felt well worth the effort, however, to attempt to determine something of the stress behavior of the embedded anchor steel, and, accordingly, a series of remote reading, electric telemeters were installed for observing the strains in the eye-bars after embedment in the concrete. The middle line of the middle chain of eye-bars in one of the cables anchored in the mass concrete of the New York anchorage was chosen for the test. Eight telemeters were installed in pairs, one on the top and one on the bottom, at

TABLE 3.—PERCENTAGE OF VARIATION FROM THE AVERAGE OF MEASURED STRESSES IN ANCHORAGE EYE-BARS FOR ADJUSTED STRANDS OF ONE CABLE AT THE NEW YORK ANCHORAGE.

Strand No.	Set No.																
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16*	17†
1	+8	+16	+19	+25	+22	+29	+31	+31	+36	+36	+34	+37	+39	+39	+40	+20	+19
2	−6	+12	+15	+18	+19	+22	+25	+23	+28	+28	+28	+29	+32	+32	+35	+19	+19
3	−1	+10	+18	+22	+20	+27	+29	+29	+34	+34	+34	+36	+36	+35	+36	+21	+18
4	−1	+19	+19	+25	+26	+25	+31	+30	+37	+35	+35	+36	+41	+39	+43	+23	+21
5	....	−4	−4	+1	+6	+8	+12	+10	+12	+11	+12	+13	+12	+12	+12	+7	+6
6	....	−19	−10	−6	−4	+1	+3	0	+3	+1	+4	+5	+7	+6	+9	+8	+9
7	....	−16	−10	−5	−3	+1	−1	+1	−1	+4	0	+4	+2	+5	+6	+4	+4
8	....	−21	−13	−7	−6	0	+1	−2	+6	+2	+2	+4	+5	+6	+6	+6	+6
9	....	....	−6	−2	+1	+4	+5	+5	+5	+8	+7	+8	+9	+10	+8	+6	+5
10	....	....	−5	0	+3	+8	+9	+8	+8	+8	+9	+11	+11	+12	+10	+6	+8
11	....	....	−14	−10	−6	−2	−6	−2	−3	+1	−1	+3	+2	+2	+4	+2	+3
12	....	....	−5	+1	+1	+8	+8	+7	+11	+8	+9	+11	+10	+15	+10	+7	+6
13	....	....	....	−11	−7	−5	−5	−3	−1	0	−2	0	+4	+3	+3	+3	+3
14	....	....	....	−17	−10	−11	−7	−9	−6	−5	0	−2	0	−1	−1	0	−1
15	....	....	....	−13	−9	−10	−7	−8	−5	−2	−5	−2	−3	−1	0	−3	−2
16	....	....	....	−21	−12	−12	−7	−10	−5	−6	−7	−3	−1	−3	−3	−3	−3
17	....	....	....	....	−11	−7	−9	−6	−7	−3	−6	−3	−3	−1	−2	+1	+3
18	....	....	....	....	−13	−14	−9	−9	−5	−6	−6	−5	−5	−3	−2	−1	0
19	....	....	....	....	−7	−7	−8	−4	−5	−2	−5	−2	−1	+2	−1	+1	+3
20	....	....	....	....	−12	−14	−8	−10	−6	−6	−6	−5	−3	−4	−4	+1	+1
21	....	....	....	....	....	−13	−14	−9	−10	−5	−6	−6	−8	−7	−5	−5	−5
22	....	....	....	....	....	−10	−8	−5	−3	−4	−3	−5	−4	−2	−2	−5	−5
23	....	....	....	....	....	−6	−12	−3	−4	+1	−2	0	−4	−2	+2	−4	−3
24	....	....	....	....	....	−12	−5	−9	−2	0	+2	+3	−4	+2	+3	−5	−2
25	....	....	....	....	....	....	−11	−7	−14	−7	−8	−7	−8	−2	−6	−2	−1
26	....	....	....	....	....	....	−12	−13	−11	−11	−13	−11	−11	−6	−9	−5	−3
27	....	....	....	....	....	....	−13	−7	−5	−4	−3	−2	−6	−1	−2	−6	−2
28	....	....	....	....	....	....	−2	−5	−2	−4	−1	−2	−2	0	+3	0	0
29	....	....	....	....	....	....	....	0	−5	0	0	+1	+2	+4	+3	+3	+5
30	....	....	....	....	....	....	....	−7	−14	−12	−11	−9	−7	−12	−9	−7	−2
31	....	....	....	....	....	....	....	0	−7	−3	−1	0	−1	+2	+1	−4	0
32	....	....	....	....	....	....	....	−9	−8	−5	−4	−3	−3	−5	+3	−2	+1
33	....	....	....	....	....	....	....	....	−20	−11	−13	−12	−12	−11	−15	−7	−2
34	....	....	....	....	....	....	....	....	−18	−12	−16	−14	−13	−16	−9	−11	−3
35	....	....	....	....	....	....	....	....	−18	−16	−18	−16	−16	−15	−12	−9	−6
36	....	....	....	....	....	....	....	....	−10	−8	−7	−7	−5	−7	−2	−5	−3
37	....	....	....	....	....	....	....	....	....	−9	−7	−7	−4	−2	−6	−1	−6
38	....	....	....	....	....	....	....	....	....	−7	−4	−8	−6	−9	−6	−1	−3
39	....	....	....	....	....	....	....	....	....	−10	−6	−8	−6	−4	−6	−4	−7
40	....	....	....	....	....	....	....	....	....	−10	−4	−8	−6	−10	−7	−3	−5
41	....	....	....	....	....	....	....	....	....	....	−3	−9	−3	−3	−3	−2	−5
42	....	....	....	....	....	....	....	....	....	....	−7	−13	−7	−7	−7	−3	−3
43	....	....	....	....	....	....	....	....	....	....	−8	−15	−13	−14	−16	−12	−10
44	....	....	....	....	....	....	....	....	....	....	−5	−9	−5	−9	−7	−5	−5
45	....	....	....	....	....	....	....	....	....	....	....	−7	−10	−7	−6	−4	−4
46	....	....	....	....	....	....	....	....	....	....	....	−7	−6	−7	−7	−1	−2
47	....	....	....	....	....	....	....	....	....	....	....	−9	−10	−8	−8	−3	−5
48	....	....	....	....	....	....	....	....	....	....	....	−5	−2	−3	−4	−2	−4
49	....	....	....	....	....	....	....	....	....	....	....	....	+3	+3	+2	+2	+1
50	....	....	....	....	....	....	....	....	....	....	....	....	−1	−2	−3	+1	−1
51	....	....	....	....	....	....	....	....	....	....	....	....	+1	−1	−4	−2	−2
52	....	....	....	....	....	....	....	....	....	....	....	....	−1	−2	−5	0	−3
53	....	....	....	....	....	....	....	....	....	....	....	....	....	−6	−5	0	−4
54	....	....	....	....	....	....	....	....	....	....	....	....	....	−6	−2	+2	0
55	....	....	....	....	....	....	....	....	....	....	....	....	....	−6	−8	−3	−5
56	....	....	....	....	....	....	....	....	....	....	....	....	....	−11	−8	−5	−6
57	....	....	....	....	....	....	....	....	....	....	....	....	....	....	−11	−4	−5
58	....	....	....	....	....	....	....	....	....	....	....	....	....	....	−18	−8	−8
59	....	....	....	....	....	....	....	....	....	....	....	....	....	....	−8	−5	−6
60	....	....	....	....	....	....	....	....	....	....	....	....	....	....	−8	−2	−4
61	....	....	....	....	....	....	....	....	....	....	....	....	....	....	−4	−1	−3

* Set No. 16, floor steel erection completed.

† Set No. 17, concrete floor-slabs in place.

points near the ends of corresponding eye-bars in the two lower tiers, the uppermost point being outside the concrete during most of the construction of the bridge. The location of the telemeters is shown in Fig. 5.

The double-cartridge, carbon-pile telemeter works on the principle that a change in the compression in a stack of carbon disks, which are part of an electric circuit, is accompanied by a change in the electrical resistance of the stack.[5] The telemeters themselves, although they are compensated and, therefore, independent of temperatures, are affected by humidity or moisture. Special precautions were taken therefore to keep the instruments dry. Each was tap-screwed directly to the top or bottom of the eye-bar as the case might be, and then a cast bronze case or cover was placed over it and independently secured to the eye-bar using a gasket between the case and the steel. Where the wires emerged from the case elastic gum and water-proofing compound stuffing was used. Before concreting, the entire assembly was wrapped with water-proofed tape, and painted. Small resistance thermometers were also attached to the eye-bars in the same cases with the telemeters for observing temperatures.

The measurements indicate quite clearly that, except for the relatively small stresses from temperature or shrinkage of the setting and curing concrete, none of the telemeters embedded in the concrete has shown any stress in the eye-bars. In other words, if the reliability of the instruments is

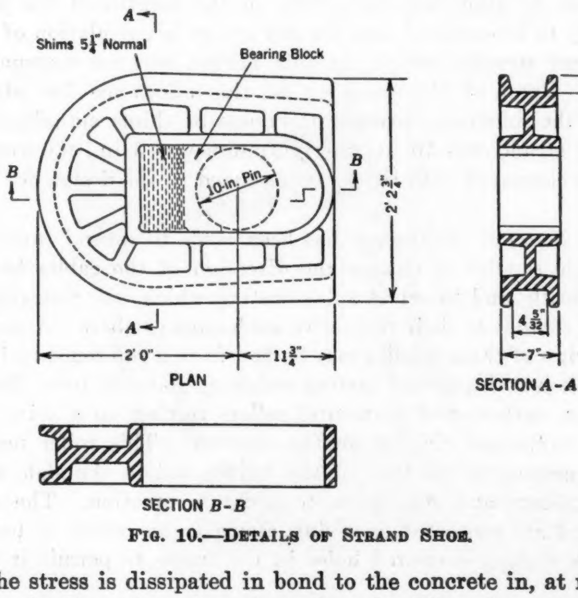


Fig. 10.—Details of Strand Shoe.

accepted, the stress is dissipated in bond to the concrete in, at most, a length of 25 ft. in the line of eye-bars investigated. There is no reason to doubt that the same is true of the stress conditions in all other bars. If these bars were carrying their full load they would at present be under a stress of 16 000

[5] "Recent Developments and Applications of the Electric Telemeter," by O. S. Peters, *Proceedings*, Am. Soc. for Testing Materials, 1927, and *Technologic Paper No. 247*, National Bureau of Standards, U. S. Dept. of Commerce.

lb. per sq. in. The telemeters are still (1932) being checked periodically in order to detect any sudden or gradual pick-up of stress, if it should occur, and can be read when any additional dead load is placed on the bridge.

Strand Shoes.—The strand shoes used for terminating the cable strands and anchoring them to the anchorage eye-bars, are steel castings of the conventional design. (See Figs. 10 and 11.) Their design was based on functional demands rather than on strength requirements. Their major dimensions were determined by "scaling up" from the corresponding dimensions of the strand shoes used on the Delaware River Bridge[6], for which full-sized tests with strain measurements were made. Afterward, a somewhat extended elastic stress analysis was applied to the design, and this indicated stresses in the shoe well within allowable limits. The detail of the shoe differs in one respect from conventional design in that the side walls of the groove restraining the strand wires are beveled rather than vertical. Although all shoes were made alike in this regard this was done primarily for the shoes in the New Jersey anchorage. At that point the strands are forced by design to bend laterally through a small angle as they leave the shoes. As each half of the strand gradually departs from the curved bearing surface of the shoe, the bevel on the sides of the groove furnishes an easement curve for the lateral deflection of the wires.

Provision was made in the design for a longitudinal adjustment of the shoe of $10\frac{3}{4}$ in. to allow for any errors in the lengths of the guide wires, quite naturally to be expected, and for any errors in calculation of the lengths of the different strands, which, in this bridge, was an unusually difficult computation. Proof of the adequacy of this allowance for adjustment is the fact that the maximum average thickness of shims actually used at the two ends of a strand was 10 in., the minimum was 3 in., the grand average being 6 in., as compared with the $5\frac{1}{4}$ in. assumed in the design for the normal thickness.

Anchorage Saddles.—Reference has been made to several functions of the anchorage cable saddles to change the direction of the cables both horizontally and vertically and to act as splay casting which will restrain and guide the individual strands to their respective anchorage eye-bars. A more detailed idea of the design of these saddles may be had from a reference to Figs. 12 and 13. Each saddle is a single steel casting weighing about 21 tons. Bearing is on a nest of 12-in. carbon-steel segmental rollers resting on a 4-in. rolled-steel slab which is supported directly on the concrete. This roller nest is keyed in the usual manner to the base of the saddle and to the slab, and is provided with side-bars and stop-plates to assist in erection. The rollers will operate dry and are protected by a dust-guard frame which is bolted to the saddle through slightly oversized holes in the frame to permit it to slide on the slab.

These rollers are required for taking up the changes in length of the back-stay cable and anchorage eye-bars due to changes in temperature and stress. The maximum calculated motion of the saddle riverward from normal,

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[6] Final Rept. of the Board of Engrs. to the Delaware River Bridge Joint Comm., June 1, 1927.

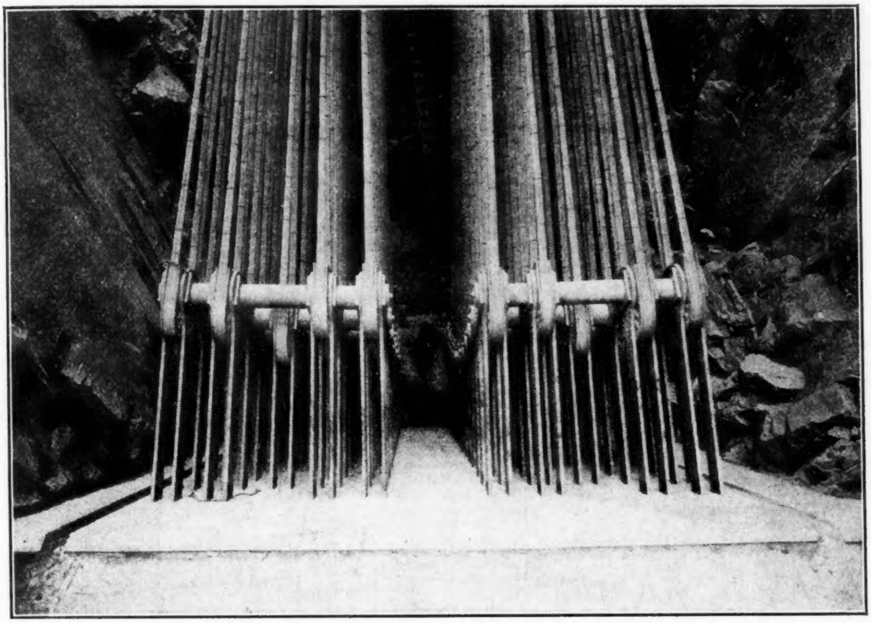


FIG. 11.—STRAND SHOES IN NEW JERSEY ANCHORAGE.

FIG. 12.—ANCHORAGE CABLE SADDLES, NEW YORK.

due to simultaneous extremes of temperature and live load, is 1.1 in., while the maximum shoreward motion from normal is 0.4 in. This latter dimension is exceeded temporarily during the construction of the cables when the saddle has to be set shoreward about 2.5 in, from normal position, which represents approximately the total back-stay stretch resulting from the addition

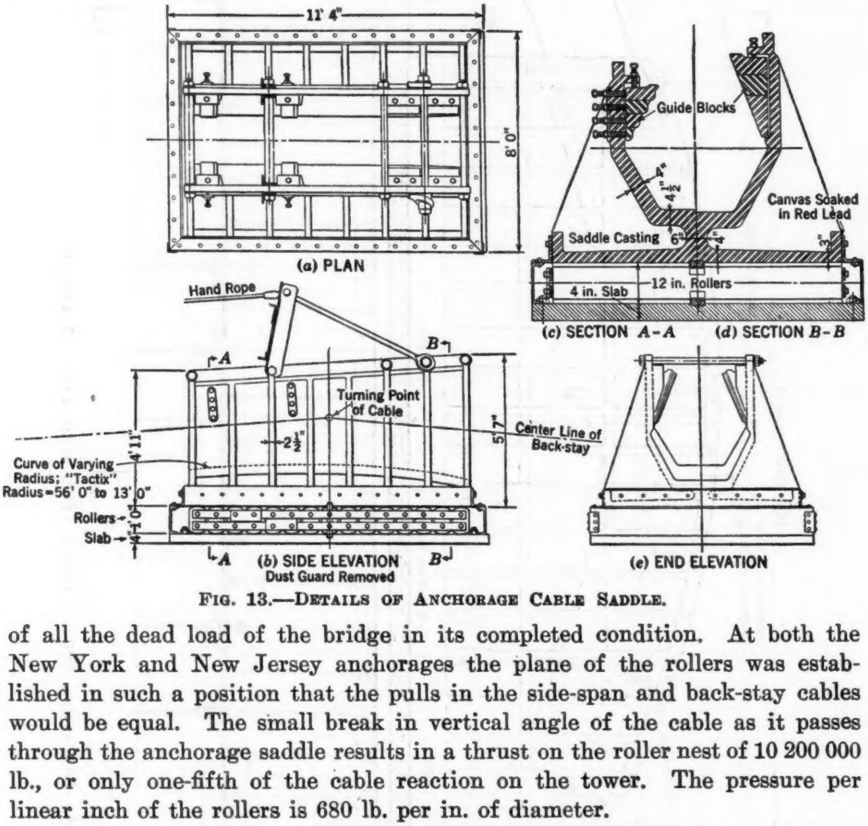


FIG. 13.—DETAILS OF ANCHORAGE CABLE SADDLE.

of all the dead load of the bridge in its completed condition. At both the New York and New Jersey anchorages the plane of the rollers was established in such a position that the pulls in the side-span and back-stay cables would be equal. The small break in vertical angle of the cable as it passes through the anchorage saddle results in a thrust on the roller nest of 10 200 000 lb., or only one-fifth of the cable reaction on the tower. The pressure per linear inch of the rollers is 680 lb. per in. of diameter.

On account of the vertical flare of the strands in the back-stay, the top strands, with their small angular deflection, have a short length of bearing on the saddle, whereas the bottom strands, with their large angular deflection, have a long length of bearing on the saddle, intermediate strands having intermediate lengths of bearing. At the upper or riverward end of the saddle all strands are in bearing while at its lower or shoreward end only the bottom strands are in bearing. Therefore, in order to equalize the pressure of the strands on the saddle along its length it was not constructed with a circular curve, but with a curve of varying radius, the greatest radius, 56 ft., being at its upper end and the smallest, 13 ft., at its lower end. To obtain uniform pressure the radius of curvature must be proportional to the number of strands in bearing; but the number of strands in bearing at various points along the saddle depends on the shape of the curve, being inversely propor-

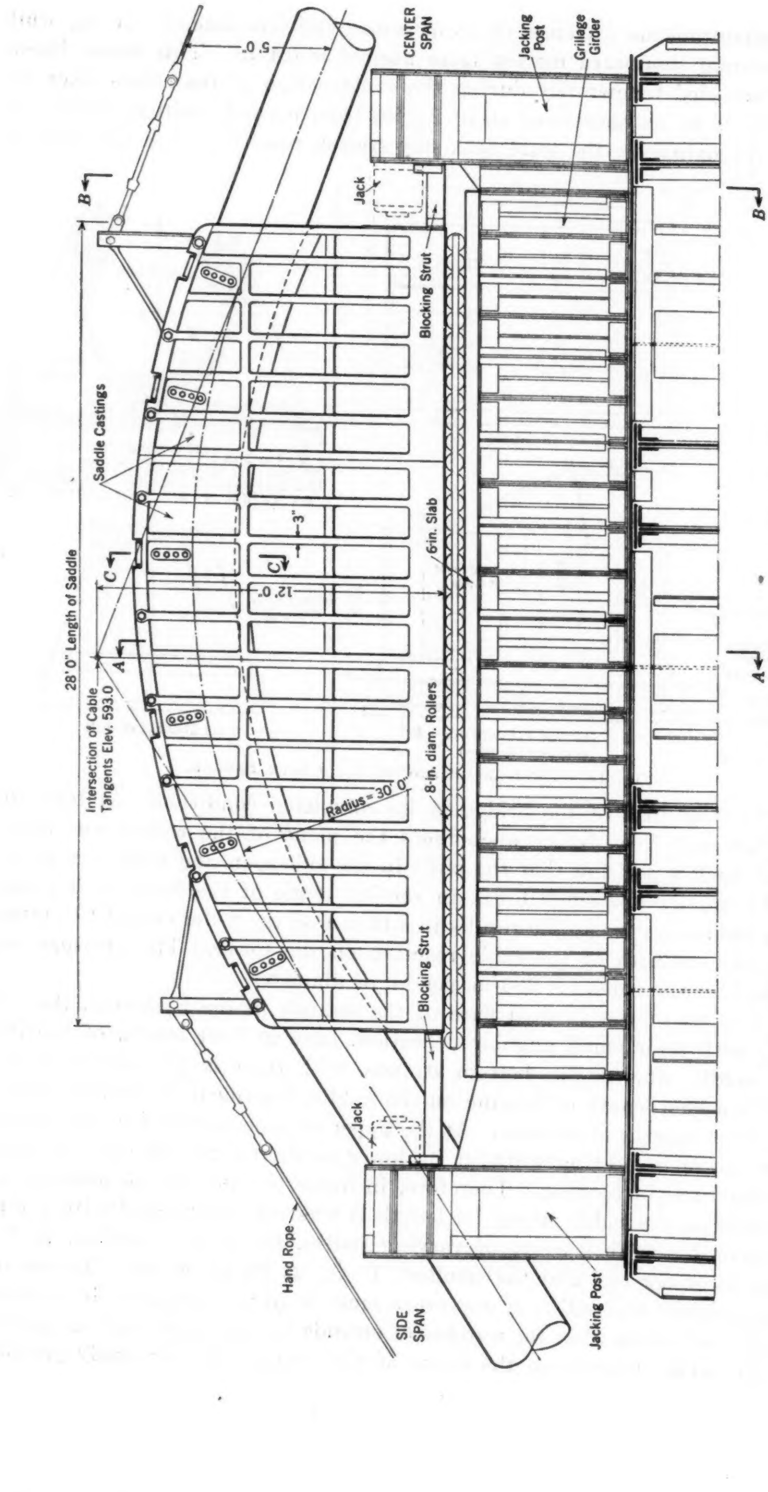


FIG. 14.—ELEVATION, TOP OF TOWER (SEE, ALSO, FIGS. 15 AND 16).

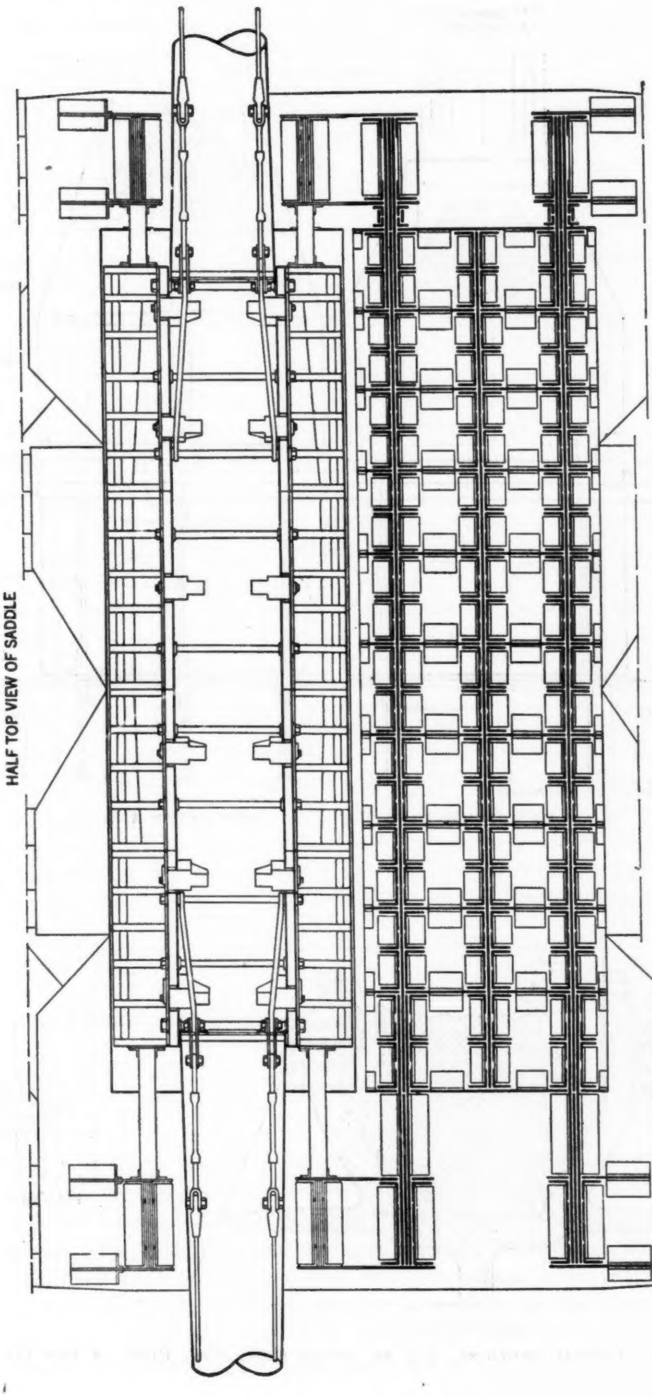


FIG. 15.—HORIZONTAL SECTION AND TOP VIEW, TOP OF TOWER (SEE, ALSO, FIGS. 14 AND 16).

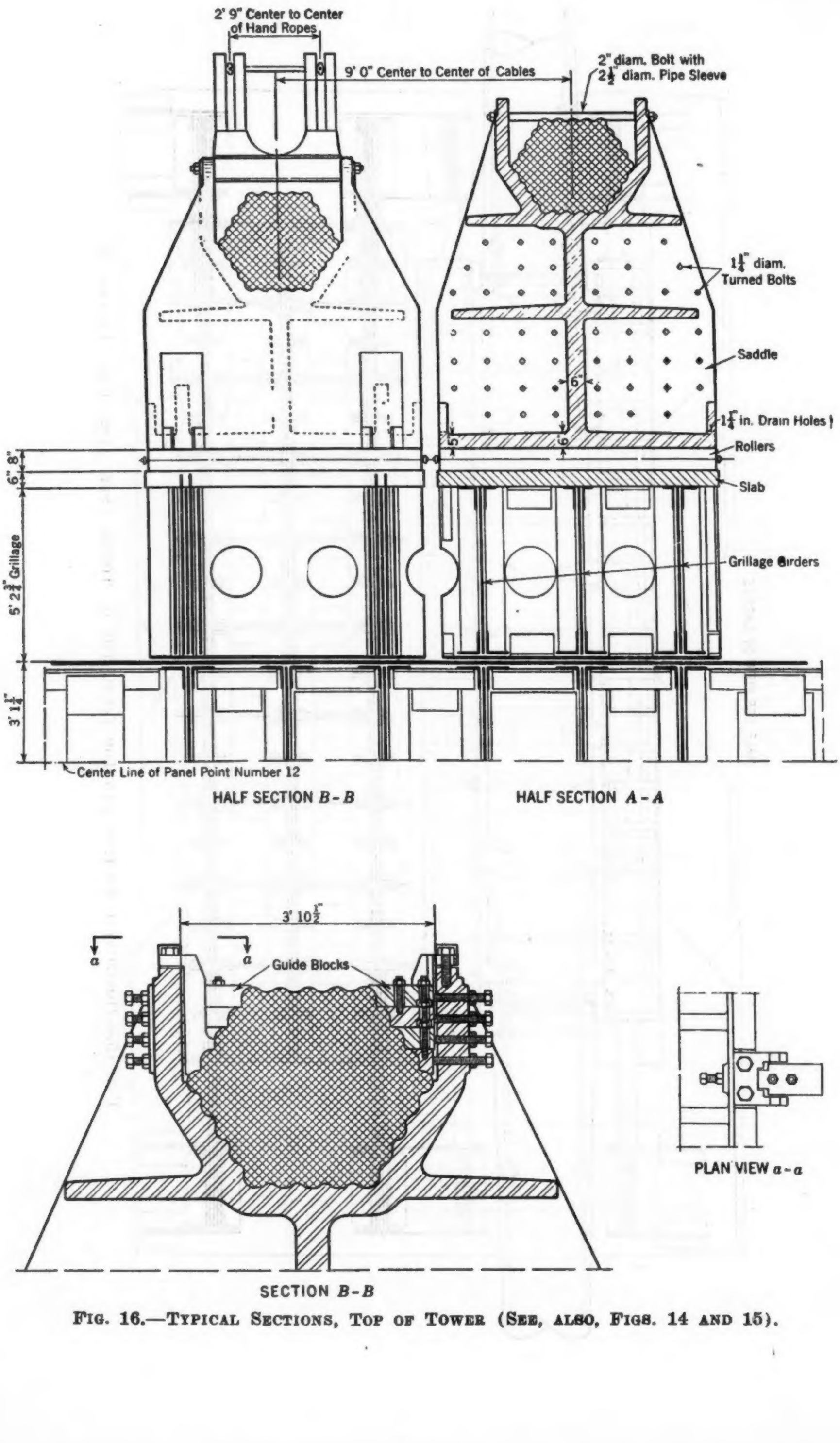


Fig. 16.—Typical Sections, Top of Tower (See, also, Figs. 14 and 15).

tional to the deflection angle of the curve. The curve, therefore, must be such that the radius varies inversely with its deflection angle. This curve is the tractix, or involute to the catenary, and by assigning a definite value to its length, and definite values to the terminal radii at each end, the parameter of the particular tractix required was determined. The bottom inside surface of the saddle was cast to this curve and, in this way, a uniform pressure on the rollers was obtained.

For about 3 ft. at the shoreward end of the saddle the sloping side surfaces flare outward in circular arcs, permitting the cable strands to splay laterally. At the top the strands are restrained and gradually led off laterally by curved wedged-shaped steel blocks. As the flaring of the strands would have made grooving of the saddle difficult, grooves were not used, and the surfaces were cast smooth and not machined.

Tower Saddles.—Where the cables pass over the tops of the towers they are supported in large saddles (Figs. 14, 15, 16, and 17). Each saddle, which is comprised of four steel castings bolted together, is 28 ft. long, 8.5 ft. wide, and 10.8 ft. high, and weighs 180 tons. The saddle rests on a bed of forty-one 8-in. steel rollers bearing on two 6-in. rolled steel slabs, butt ended together. These slabs are tap-bolted from beneath to the top flanges of steel grillage girders. There are three grillage girders, 5 ft. $1\frac{3}{4}$ in. deep, with closely spaced transverse diaphragms, under each saddle. The grillages serve to keep the cables clear of the top of the tower and assist in distributing the load to the tower columns. Jacking posts are connected to the ends of grillages.

As previously mentioned the rollers are provided not for the purpose of permitting the saddles to roll on the tops of the towers for live load and temperature saddle motions, but to facilitate erection and to permit future adjustments. Under ordinary conditions the saddles will be blocked against the jacking posts of the grillages in such a way as to prevent relative motion between saddle and tower, and the saddle motions will be participated in by the tower; that is, the tower will be forced to deflect longitudinally.

TABLE 4.—SADDLE MOTIONS AND CABLE DEFLECTIONS

Item of dead load	Motion of Saddles, in Inches		Increase of Center-Span Sag, in Feet	
	Additional	Total	Additional	Total
Upper deck steelwork	....	8.00	...	7.0
Concrete slabs—side roadways	5.25	13.25	4.3	11.3
Concrete slab—center roadway	2.00	15.25	1.9	13.2
Lower deck complete	7.75	23.00	6.8	20.0

During the construction of the cables there is no appreciable movement of the saddles, except that due to temperature, because each strand, as it is added to the cable, has the same stress as in the completed cable. The addition of the dead load of the floor during its erection, however, increases the stress in the cable, and the consequent stretching of the side-span cables causes the saddles to move riverward. The magnitude of these saddle motions for the various increments of dead load are given in Table 4, which also

gives the increase of the sag of the cables in the center span caused by the combined effect of the stretch of the center-span cables and the movement of the saddles.

As it was considered desirable to have the saddles in their normal position over the center line of the tower, and thus avoid eccentricity of load, when the bridge is completed in its final stage with the lower deck, the saddles were set on the rollers 23 in. shoreward from this position during the construction of the cables. Rather than permit the saddles to roll riverward as the dead load was added (which would have been an uncertain action on account of the rolling friction and the flexibility of the towers), the saddles were kept blocked to the towers during the erection of the floor. Consequently, the tower tops were deflected riverward as the saddles moved, and at intervals during the floor erection the blocking was temporarily removed and jacks were operated between the saddles and the towers to roll the tower tops shoreward and thus avoid an undesirable amount of bending of the towers. This jacking was done twice during steel erection and again after the side roadway slabs were poured. In the last jacking operation the tower tops were rolled to a position, 2 in. shoreward from a vertical position, and the permanent steel blocking was placed. This position was selected so that the towers would be vertical when the upper deck is completed by the paving of the center roadway, and the saddles move 2 in. riverward.

At such future time as the lower deck is constructed the saddles will move riverward the remaining $7\frac{3}{4}$ in., to their final position, and the tower tops will be deflected that amount. The tower tops are then to be jacked back to a vertical position, and beyond to a position $5\frac{1}{2}$ in. shoreward, in order to minimize the stresses in the tower. The bending of the tower due to the lengthening and shortening of the side-span cables by live load and temperature is greater in the riverward, than in the shoreward, direction, and the riverward bending occurs with a greater vertical load on the tower. Consequently, to minimize the combined stress from vertical load and bending, the maximum stress on the riverward and shoreward sides of the tower must be equalized, which is accomplished by having the tower bent shoreward for the condition of dead load and normal temperature.

Thus, it is seen that the rollers have functioned only three times during the construction of the upper deck and will function again only when the position of the tower tops is adjusted after the lower deck is constructed.

Reference to Fig. 14 will show the very simple form of roller nest used. The usual side-bars are included, but there is no provision for keying the rollers to either the saddle or the slab, the pressure itself being considered ample insurance of maintaining the alignment.

The large jacking posts that are attached to, and form a part of, the grillages at either end, are designed to withstand the maximum horizontal tower reactions and the maximum jacking forces required during any stage of construction of the bridge. Finished bosses have been made on the ends of the saddles to receive the concentrations from the jacks.

The maximum reaction on each saddle is 55 600 000 lb. This force requires a long bearing surface between the cable and the saddle, and must

FIG. 17.—TOWER SADDLE, GEORGE WASHINGTON BRIDGE.

be distributed more or less uniformly to the tops of the four inside tower columns by means of the saddles and grillage girders. The girders, including the jacking posts, were made to extend the full width of the tower, and the saddles were made long enough to permit a 30-ft. radius for the cables, and still allow room between the ends of the saddles and the jacking posts for the jacks and the necessary saddle motions. The cables enter the saddles at angles with the horizontal of about 33° on the side-span side and 20° on the center-span side, resulting in unsymmetrical saddles.

This inequality of angle, results in a corresponding inequality in tension in the cable at the two ends of the saddle, as previously mentioned, the tension being 12% greater on the side-span end. The difference in tension is absorbed by friction between the cable and the saddle, the amount of friction required being 13% of of the vertical load. This is increased to 15% under the most adverse conditions of live load and temperature.

For the cable to be safe against slipping in the saddles, the actual amount of friction available must exceed the amount required. Friction tests of wires, made in connection with the Delaware River Bridge,[6] indicate that a friction between wires and saddle of 20% can be counted on and this figure was used in the design, especially as it related to conditions during erection. In order to obtain the maximum friction, the inside bearing surfaces of the saddles were roughened after they were machined, and instead of a weather protection of grease they were given a coat of paint. It was expected that, if anything, the paint would increase the friction, but experiments made in the field after the first strand had been set in the saddle, showed that this was not the case and, consequently, the paint was removed.

While the first strands were being adjusted under low temperature, some of them slipped in the saddles, and, by making the necessary field observations and calculations for the conditions at the time, this slipping was used to ascertain the actual coefficient of friction. It was found that slip occurred when the unbalanced pull was 23% of the vertical load for a painted surface, and 30% for a surface from which the paint had been removed, which showed that the assumed value of 20% for the friction of a strand in the saddle was conservative.

The completed cable has a greater resistance to slip than that due merely to friction on the saddle. Before the cable can slip, it must bend and unbend as a whole at the two ends of the saddle, and for this to occur all the wires in the cable must slip on each other throughout the length of the saddle, overcoming the friction between them. The internal friction of the wires thus tends to lock the cable into its curved shape in the saddle and adds greatly to the factor of safety against slip.

Regarding the more detailed design of the tower saddles involving the determination of thicknesses of metal used, little can be said other than that it was based upon a conservative application of general and simple rules of design. The actual distribution of stress throughout such a casting is too indeterminate to permit of an application of the law of elasticity. The details of the saddles, such as tie-rods and wedge-blocks, were more or less conventional.

However, in dimensioning the cross-section of the inside surfaces of the saddle and the shape of the strand grooves, an allowance was made for the tendency of the strands to flatten under their own weight, when placed in the saddle, in spite of their seizing. Consequently, instead of assuming truly circular strands assembled to form a regular hexagon, it was considered that each strand would flatten to the shape of an ellipse, 4 by 5 in., and that the entire cable would take the form of a flattened and elongated hexagon.

Cable Bands.—The cable bands, on which the wire-rope suspenders bear, are in the form of two semi-circular steel castings bolted together in such a manner that they clamp the cable tightly between them. Each band carries in grooves cast in the outside periphery the bights of two suspender ropes. Fig. 18 shows the largest band, which is used at all panel points in the side

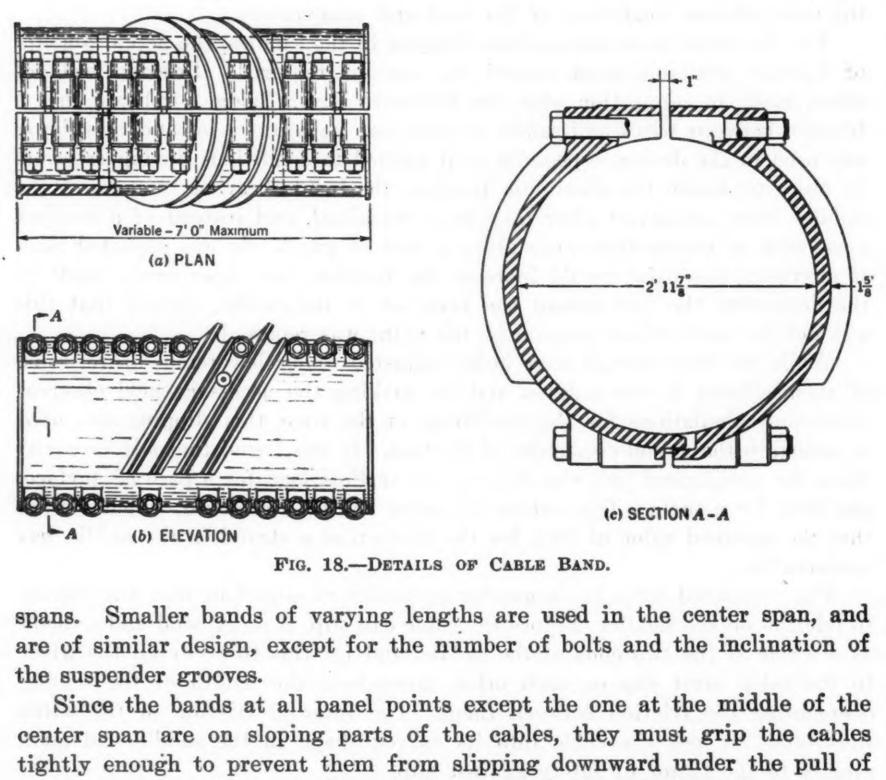


FIG. 18.—DETAILS OF CABLE BAND.

spans. Smaller bands of varying lengths are used in the center span, and are of similar design, except for the number of bolts and the inclination of the suspender grooves.

Since the bands at all panel points except the one at the middle of the center span are on sloping parts of the cables, they must grip the cables tightly enough to prevent them from slipping downward under the pull of the suspenders. The total clamping force provided is calculated for any particular band by dividing the component of the maximum suspender pull that is parallel to the cable by the allowable coefficient of friction. The component of suspender pull acting normal to the cable is not considered in the design as effective in creating friction. The required clamping force divided by the allowable stress in the bolts gives the required number of bolts. Heat-treated, turned bolts, $2\frac{3}{8}$ in. in diameter, were used at an allow-

able stress of 100 000 lb. per bolt, equivalent to a unit stress of 29 000 lb. per sq. in. of net section. A coefficient of friction of 15% was assumed in determining the necessary clamping forces.

The determination of what constitutes a safe allowable coefficient of friction between the bands and the cables is a mooted point. Laboratory tests of the ultimate resistance to slipping were made by the Delaware River Bridge Joint Commission,[6] on a band clamped to a sample section of the 30-in. cables. These tests showed an average coefficient of friction of about 60 per cent. It is believed that the local contraction in diameter of the cables due to the relatively greater compaction under the band than elsewhere is accountable for this high value. Tape measurements of circumferences of the cables of the George Washington Bridge for short distances adjacent to the bands, verified this relative swelling of the cables at both ends of the bands. These measurements showed a cable diameter appreciably less at the immediate ends of the band than at points 6 to 18 in. away. With such conditions existent, a 60% ultimate value of friction seems reasonable, fully justifying an allowable value of 15 per cent.

To insure, further, the development of friction between the bands and the cables the inside machined surfaces of the bands were specified to be finished to a definite degree of roughness (16 transverse tool cuts per inch). The use of grease on these machined surfaces for protection was discarded in favor of the application of one shop coat of paint.

The thickness of the main shell of the bands was made $1\frac{5}{8}$ in. The shell is subjected not only to the ring tension developed in tightening the bolts, but also to considerable bending stress because the bolts are eccentric to the shell, and because the band must deform slightly from its truly circular shape in the process of squeezing the cable to fit it. It was necessary for the shell to have a margin of strength to take bending stresses and, at the same time, to be thin enough to stand some deformation.

During construction, while floor steel was being erected, a longitudinal crack developed in one of the cable bands in the center span. This crack, at the mid-depth of the band, extended about 14 in. in from one end and was considered serious enough to warrant replacement, which was made before the addition of the concrete slabs. A series of strain measurements were made on the new band to show the actual stresses as the bolts were tightened and during the addition of subsequent floor loads. The circumferential strains on the outside periphery of the main shell indicated stresses well within the allowable, but the strains in the extreme outside fibers of the end ribs of the band showed stresses above the yield point of the material. Longitudinal strains due to beam action from the suspender concentrations indicated that these stresses were very small. Fig. 19 shows the results of these measurements. It was evident that the additional stiffness provided by the end ribs resulted in an overstress in the latter as the band was deformed in squeezing the cable into a circular shape. Inasmuch as this overstress was local, and was immediately relieved by a redistribution of stress on adjoining sections, it was not considered harmful; a close inspection of all the remaining bands failed to disclose any further cracks either at this time or later.

It is quite possible that a local defect may have caused the crack, although none was detected in a superficial examination or chemical analysis of borings.

In accordance with the usual practice the bands were designed with the joints between halves placed vertically; that is, with the bolts in a horizontal position. A nominal gap of 1 in. was allowed between flanges for clearance

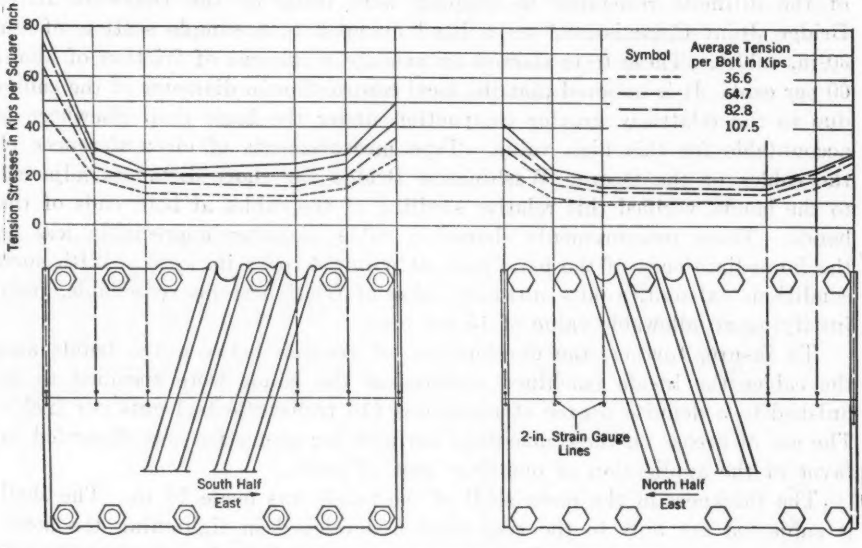


FIG. 19.—MEASURED STRESSES IN CABLE BAND.

and caulking. Circumferential grooves were provided on the inside surfaces of the longer bands to give clearance for the permanent seizing of the cables at these points. The ends of the bands were counter-bored in the usual manner for terminating the cable wrapping, and for caulking. The bolts and bolt housings were made quite long, so that the bolts could be close to the cable and their eccentricity minimized. The use of long bolts also lessened the effect on bolt tension of differences in temperature of the cable and the band. All bolts were provided in the shop with holes for strain-gauge measurements which were relied upon for the final check of the required bolt tensions. Calibrated dynamometers, checked frequently with the strain-gauges, were used in tightening the bolts.

To determine the exact diameter of bore for the cable bands the contractor was obligated to construct a full-sized sample section of cable 10 ft. long, and to compact it in the same manner as he expected to compact the actual cables. This gave a diameter of 35⅞ in., which value was used in the final details.

Special bands are used adjacent to all saddles for terminating the wrapping and for providing means of attaching housings for the unwrapped parts of the cables at these points.

Suspenders.—The suspenders were designed to carry to the cables the entire dead weight of the completed upper and lower decks and a local live

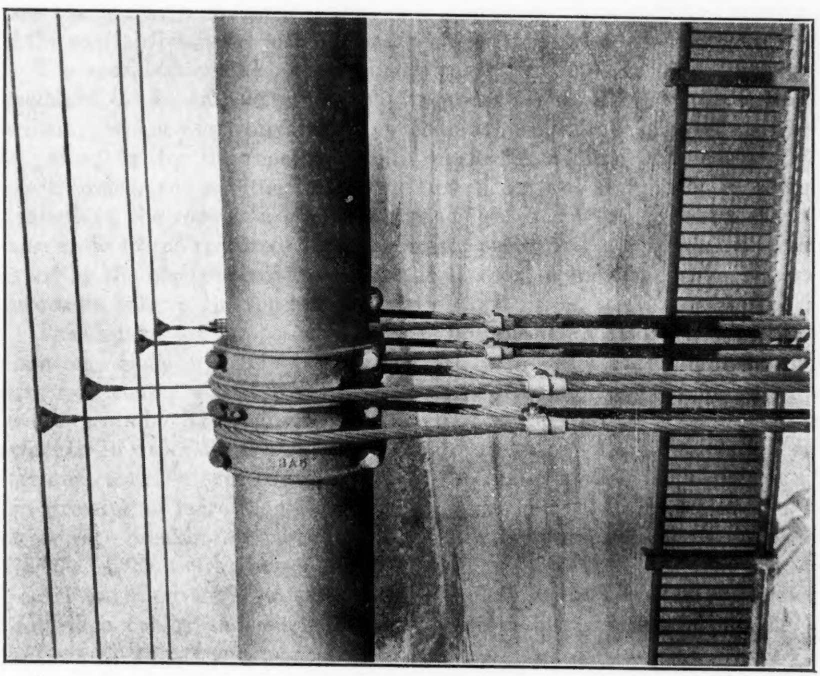


FIG. 20.—VIEW OF UPPER SUSPENDER CONNECTION.

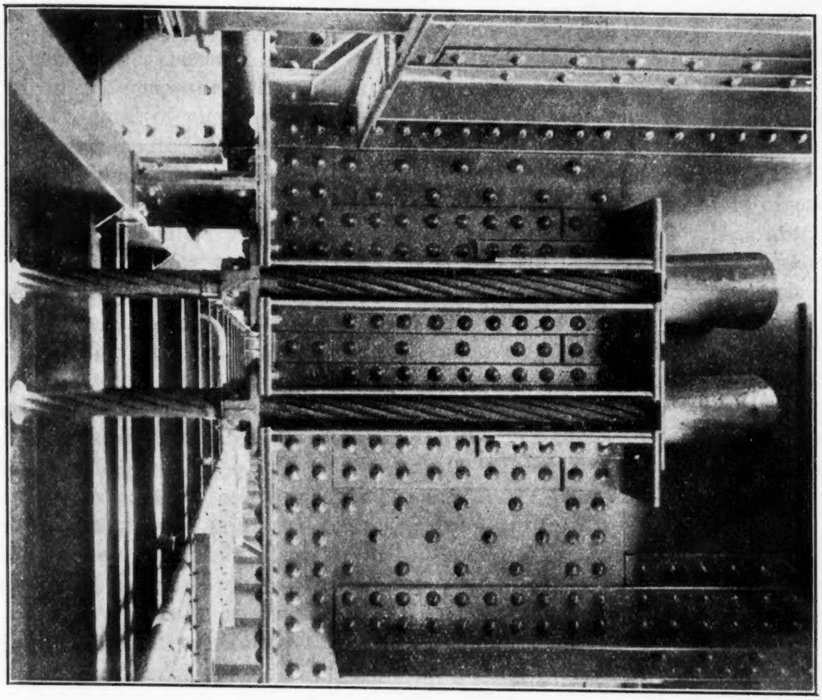


FIG. 21.—VIEW OF LOWER SUSPENDER CONNECTION.

load distributed to them by the stiffening trusses. Each suspender consists of a single wire rope, $2\frac{7}{8}$ in. in diameter, looped over the cable in grooves in the cable band, socketed at both ends, and attached directly to the floor-beam. Two of the suspenders are supported by each cable at a panel point, which makes a total of eight suspenders, or sixteen lines of rope per panel. The maximum length of suspender is 685 ft. and the minimum, 48 ft., and the total length of suspender rope in the entire bridge is 170 000 lin. ft.

At a point 5 ft. below the center line of the cable the two halves of each suspender are pulled in from the 42-in. diameter, in which they pass around the cable band, to a spacing of $14\frac{1}{2}$ in. and are held together by an articulated tension tie, which is prevented from slipping down by zinc collars cast on the ropes. From this point to their connections on the floor-beams the ropes hang vertically. In this way, the amount of space occupied by the suspenders at the floor level is kept to a minimum. Fig. 20 shows the design of the articulated tension tie and its assembly on the ropes.

The calculated maximum stress in the two parts of a suspender is 280 000 lb., or 35 000 lb. per sq. in. Of this, 200 000 lb., or 72%, is due to dead load and the remainder to live load and temperature. The suspender ropes were specified to have a minimum ultimate strength of 1 200 000 lb. in two parts, over a sheave equal in diameter to the band. Therefore, a safety factor of more than four is provided in the suspenders.

Using live loads of varying length and intensity according to the design specifications it was found that the maximum stress in a suspender occurs with the extended live load of 4 000 lb. per ft. over the entire bridge combined with a partial load of 7 700 lb. per ft. over a length of 360 ft., and with the lowest temperature. For this condition the suspenders carry 81% of the partial live panel load, and all the dead and extended live panel load.

The specifications also required the suspender ropes to have a maximum stretch of 0.3 in. in a length of 100 in. under a pull of 200 000 lb. after pre-stressing, which was equivalent to a modulus of elasticity of about 17 000 000 lb. per sq. in. for the rope as actually made. A nominal diameter of $2\frac{3}{4}$ in. was specified, but not the total wire area. Nor was the minimum ultimate strength of the rope in a single part specified, except that it was to have the same ratio to the required strength, in two parts, over a sheave as that established by the earlier acceptance tests, if the contractor chose to make acceptance tests on straight ropes thereafter.

The contractor proposed and was permitted to use ropes $2\frac{7}{8}$ in. in diameter, made up of 6 strands of 37 wires each and an independent wire rope center with 7 wires in each of its 6 outside strands and a 19-wire center strand. The individual wires in the rope are of varying size and strength in order to give a rope of high efficiency. Each rope has a cross-sectional metallic area equal to 4.05 sq. in. and a modulus of elasticity after pre-stressing of more than 18 000 000 lb. per sq. in. The acceptance tests gave an average breaking strength of 1 336 000 lb. for two parts over a sheave, and 778 000 lb. for a straight rope.

All the rope used for suspenders was first utilized by the contractor as footbridge cables and was pre-stressed before being measured to the long

lengths required for that purpose. In service in the footbridges the rope was permitted to be stressed as high as 230 000 lb., or 65% more than the maximum design stress of the suspenders. It was felt that this higher stress rather than harming the rope, would be advantageous as additional pre-stressing.

When the cable spinning was completed, the ropes were released from carrying the footbridges; then, successively, they were hung free under a condition of balanced horizontal tension at the towers, with a predetermined sag in the center span, and were measured off into the proper lengths for suspenders. A detailed program had been prepared giving the location along the ropes of every one of the suspenders to be cut from them, and the lengths to be measured. To calculate these lengths, the final dead load lengths, as determined from the final elevations of the cables and floor at each panel point, had to be corrected to take care of the difference between the final dead load stress in the suspenders and the stress in that part of the ropes from which each suspender was to be cut. A further correction of length was made for any differences that had been observed in the relative elevations of the four cables as erected. This schedule was revised from time to time, as the work progressed, to meet the unavoidable re-arrangements necessary in actual field operation.

The design did not contemplate the use of shims for adjusting the lengths of the suspenders. Instead it was believed that, with special care in the manufacture, measurements, and erection of the ropes, adjustments would become an unnecessary refinement. With this in mind the specifications required a uniform modulus of elasticity for the rope with a maximum variation of 10 per cent. To eliminate changes in length due to twists subsequent to measuring, paint marks were required to be placed along the ropes when they were being measured, and the contractor was also obligated to have these paint marks in line when the floor-beams were hung upon the suspenders.

As stated previously, the suspenders are socketed for attachment to the floor-beams. The sockets, shown in Figs. 21 and 22 are very simple in design. They are made of cast steel in the form of truncated cones, $7\frac{1}{2}$ in. to 9 in. in diameter, and $12\frac{1}{2}$ in. long, cored out in the usual way for the "broomed" wires and the zinc spelter. They are drilled near the bottom for two 0.5-in. steel pins which project on the inside into the broomed wire ends and thus prevent the socket from turning on the rope or slipping down the rope during handling and before the erection of the suspenders. In as many cases as possible the same sockets that were on the ends of the ropes when they were in the footbridge cables were used without removal from the rope.

Cable Wrapping.—The cables are protected by a wrapping of soft, annealed, and double-galvanized steel wire, continuous between cable bands, the cables being painted before and after the wrapping was applied. No. 9 (B. W. G.) wire, with a diameter of 0.151 in. over galvanizing, was used. It has an ultimate strength of 1 100 lb. and a yield point of 600 lb. per wire. The contractor was not permitted to wrap the cables until after the dead load of the entire steel floor deck and concrete roadway and sidewalk slabs were

in place, or, in other words, until the bridge was practically completed in its initial traffic condition with two-thirds of the final dead load in place.

Because of the unprecedented size of the cable, and because the addition of dead load in the future will increase the cable stress, tending to decrease

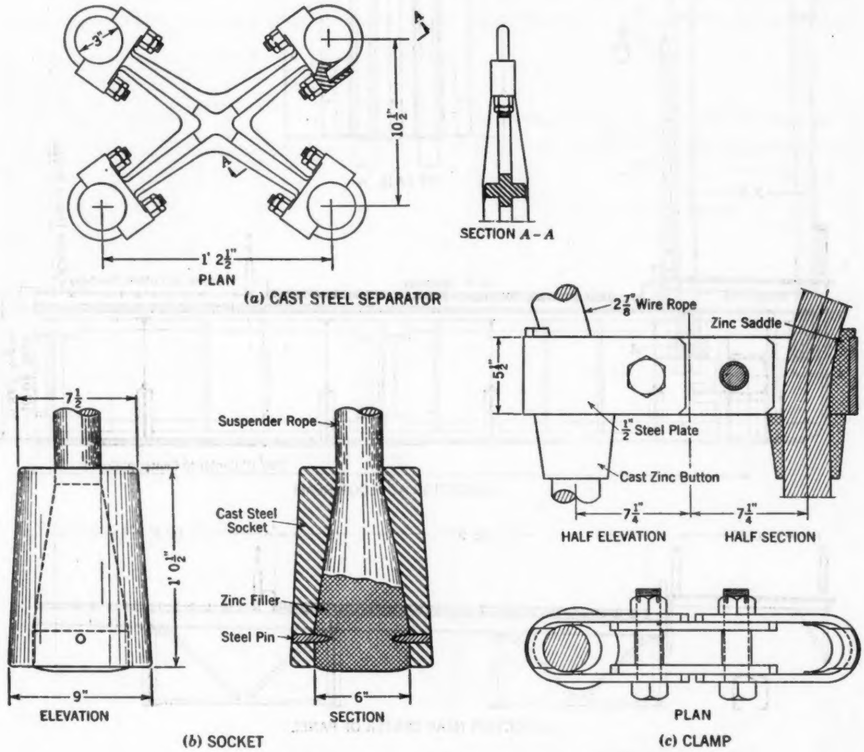


FIG. 22.—DETAILS OF SUSPENDER ROPE FITTINGS.

the diameter of the cable and loosen the wrapping, a tension of 400 to 500 lb., considerably greater than that used for previous large bridge cables, was specified.

DESIGN OF FLOOR SYSTEM

General Description.—The floor system of the upper deck, as constructed initially, is composed as follows (see Figs. 23 and 24): The main floor-beams, hung from the cables by the suspenders, are spaced 60 ft. apart. They support eight lines of roadway stringers and two fascia girders at their ends. On top of the stringers are placed transverse or secondary floor-beams, spaced about 5 ft. apart, which carry the concrete roadway slabs and four lines of steel curbs. The sidewalk slabs are supported by transverse beams, spaced 3 ft. 9 in. apart, which frame between the fascia girders and longitudinal beams fastened at 20-ft. intervals to the outside roadway stringers. Two

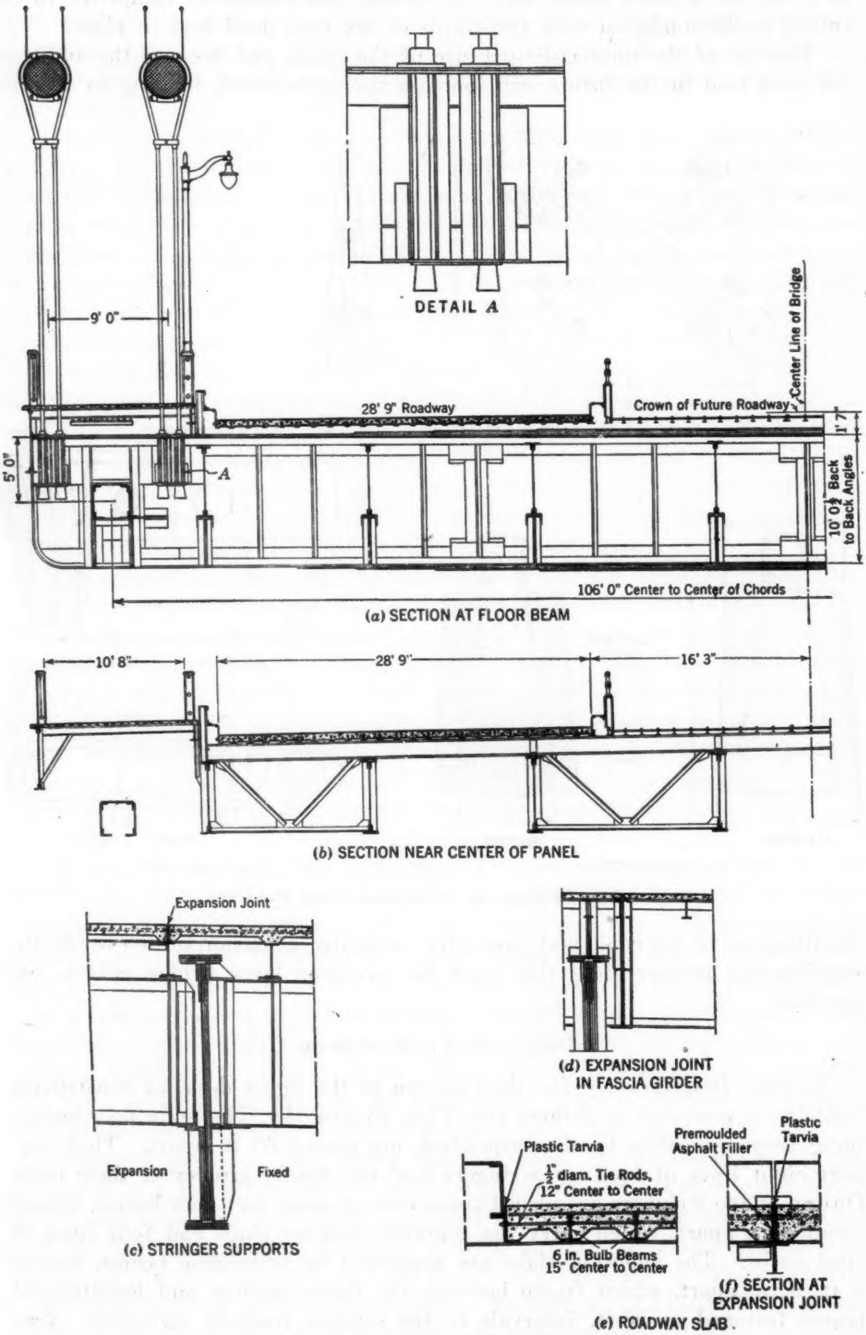


FIG. 23.—TYPICAL FLOOR DETAILS.

continuous wind chords, spaced 106 ft. apart, pass through holes in the floor-beams and, together with the main laterals placed just under the roadway stringers, form the wind trusses. These chords also form the top chords of the future stiffening trusses. The stringers and fascia girders are provided with expansion connections, and the roadway and sidewalk slabs with expansion joints on one side of each main floor-beam to allow for the angular

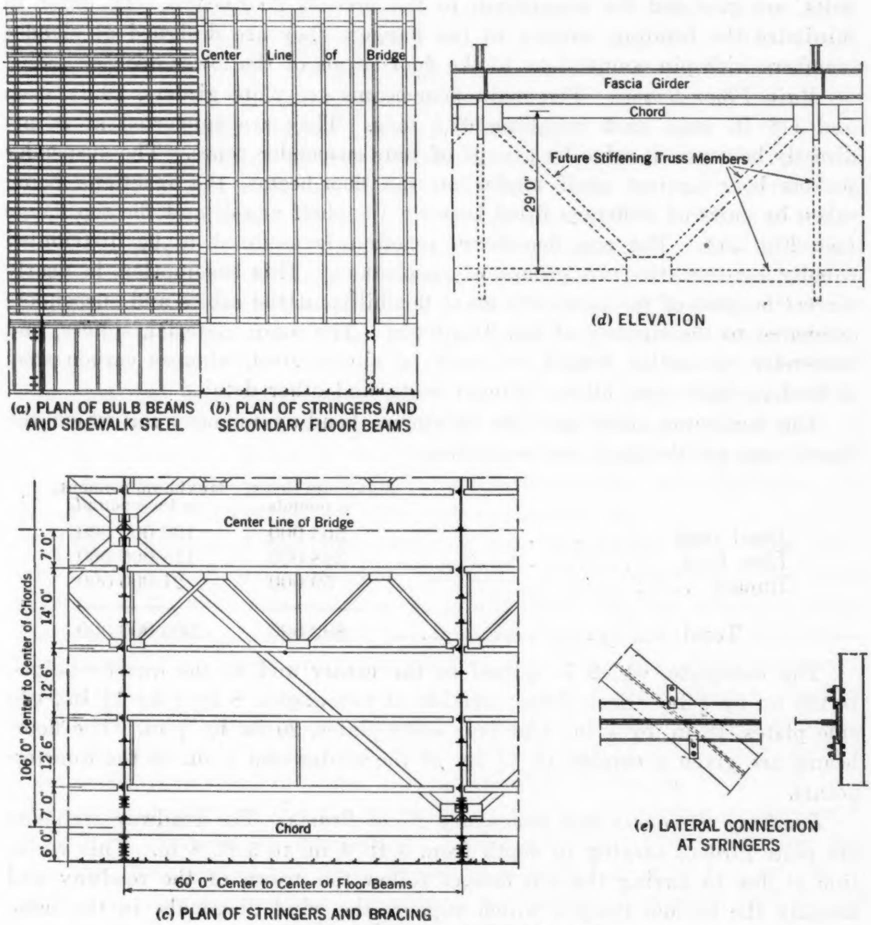


FIG. 24.—TYPICAL FLOOR FRAMING.

distortions of the cables and floor from live load, only the wind chords and laterals being continuous past the floor-beams. Under the initial condition, without stiffening trusses, the angular motion centers at the wind chords. When the trusses are added the motion will center at their neutral axis. The articulation given by these joints will serve to eliminate participation by the floor structure in the stiffening truss action and will be an advantage in the erection of the lower deck.

The design for the lower deck, as shown in Mr. Ammann's paper[7] previously mentioned, is tentative and was developed only far enough to determine the design dead load, provision for future connections, and depth of stiffening trusses. The lower deck floor-beams will be suspended from those of the upper deck by means of structural hangers which will also form verticals in the future stiffening trusses. Rivet holes, temporarily filled with bolts, are provided for connection to the present floor-beams. In order to minimize the bending stresses in the hangers they are designed as slender members with pin connections to the floor-beams of the lower deck.

Main Floor-Beams.—The main floor-beams are plate girders, 10 ft. deep and 118 ft. long, each weighing 62.5 tons. They are supported at points directly below each cable by groups of four suspender ropes. The suspender sockets bear against shelf angles on the floor-beams, the reactions being taken by pairs of stiffeners fitted between the shelf angles and the top flange (see Fig. 21). The end floor-beam reaction is assumed to be distributed equally between the two groups of suspenders. This assumption is nearly correct because of the relatively great flexibility of the cables and suspenders compared to the rigidity of the floor-beam. The main material, splices, and suspender connection details are made of silicon steel, whereas carbon steel is used for stiffeners, fillers, stringer seats, and other details.

The maximum shear and the maximum bending moment for which the floor-beams are designed, are as follows:

	Maximum shear, in pounds	Maximum moment, in inch-pounds
Dead load	505 000	166 000 000
Live load	328 000	118 000 000
Impact	59 000	21 000 000
Total	892 000	305 000 000

The web-plate, which is spliced at the center and at the quarter-points, is 120 in. by $\frac{1}{2}$ in. Each flange consists of two angles, 8 by 8 by $1\frac{1}{8}$ in., two side plates, 18 in. by $\frac{1}{2}$ in., and four cover-plates, 20 in. by $\frac{3}{4}$ in. The floor-beams are given a camber of $1\frac{1}{4}$ in. at the center and 1 in. at the quarter-points.

Roadway Stringers and Secondary Floor-Beams.—The roadway stringers are plate girders varying in depth from 5 ft. 4 in. to 5 ft. 8 in. This variation is due to having the top flanges follow the crown of the roadway and keeping the bottom flanges, which support the wind diagonals, in the same plane. The flanges are made of silicon steel, and the web-plates (which are $\frac{3}{8}$ in. thick), and all details are made of carbon steel. Each flange consists of two angles, 6 by 6 in. by $\frac{1}{2}$ in., with a cover-plate, 14 in. by $\frac{1}{2}$ in., for the four center stringers, a cover-plate, 14 in. by $\frac{3}{4}$ in., for the next two outside stringers, and no cover-plate for the outside stringers.

Both the fixed and expansion ends of the stringers have seated connections to the floor-beam, the expansion end being provided with bolts in

[7] *Proceedings*, Am. Soc. C. E., August, 1932, p. 1010, Fig. 20.

slotted holes. At the fixed ends, light connections are also made at the top to hold the floor-beam and the stringers from tipping. At the expansion ends and at the center of the panel, the stringers are held together in pairs with cross-frames, and a light horizontal bracing is used in the plane of the top flanges between the two central pairs of stringers.

The secondary floor-beams are 16-in., 43-lb. I-beams, spaced 5 ft. 2 in. apart, except at the expansion joint in the roadway slab where a 16-in., 50-lb. I-beam is used, with a spacing of 4 ft. 2 in. on either side. The beams follow the crown of the roadway, except that those in the middle bay under the curved part of the roadway crown are straight. They are riveted to the tops of the stringers and have beveled fillers under their ends to take care of the slope. The beams in the three middle bays, which are 14 ft. 0 in. wide, are made of silicon steel, and those in the four outer bays, which are 12 ft. 6 in. wide, are continuous over two spans and are made of carbon steel.

In designing the secondary floor-beams only 75% of the wheel loads was assumed to be carried by any one beam, the other 25% being distributed to adjacent beams by the floor-slab. A careful analysis of the action of the slab on its yielding supports showed that this assumption was conservative.

Roadway Curbs and Paving.—Four lines of steel curbs, of the double-step type (see Fig. 25), are supported by the secondary floor-beams. The two outside curbs are 90 ft. apart in the clear, but this space is broken up into three roadway spaces by the two intermediate curbs which are located so as to give a clear width of 28 ft. 9 in. for the two side roadways. At present (1932), only these two side roadways are paved.

The lower step of the curbs is 10 in. high and 12 in. wide. The upper step is 18 in. high and 7 in. wide. The outside curbs are close to the inner railings of the sidewalks which serve as additional barriers, while the intermediate curbs are provided with two-line pipe railings for the same purpose. The intermediate curbs are constructed so that when the center roadway is paved, the top step and railing can be removed and a plate added to the back of the lower step, in this way giving a 30 ft. 6-in. center roadway, separated from the side roadways by 10-in. curbs, 12 in. wide. If it is decided to have two roadways instead of three, the intermediate curbs can be moved from their present position and the two lower steps placed together at the center, for which arrangement connections to the secondary floor-beams are now provided, thus giving two 44-ft. roadways separated by a 10-in. curb, 24 in. wide. Another alternative is to remove the intermediate curbs entirely and have a single roadway 90 ft. wide.

The roadway pavement consists of monolithic reinforced concrete slabs, 28 ft. 9 in. wide between curbs, and 60 ft. long between expansion joints. The main reinforcing consists of 6-in., 14-lb. bulb beams running longitudinally, spaced 15 in. on centers, and riveted to the secondary floor-beams. The transverse reinforcing consists of $\frac{1}{2}$-in. tie-rods, 12 in. apart, located 4 in. above the bottom of the bulb beams, and $\frac{1}{2}$-in. reinforcing rods, 6 in. apart, placed on top of the beams and tied together with $\frac{1}{2}$-in. longitudinal rods spaced midway between the beams. The top of the slab is $2\frac{1}{2}$ in. above the top of

the bulb beams and the bottom is haunched 3 in. between the beams. The thickness of the slab is, therefore, $8\frac{1}{2}$ in. at the beams and $5\frac{1}{2}$ in. between the beams, the average thickness being 7 in.

The slabs have expansion joints, $\frac{3}{4}$ in. wide, over the secondary floor-beam on the expansion side of each main floor-beam. On the expansion side of this joint **T**-clips, fastened to the under side of every other bulb beam, project under the top flange of the secondary floor-beam to hold the slab down. A pre-moulded asphalt filler, $6\frac{1}{2}$ in. deep, is placed in the joint, and the upper 2 in. of the joint are filled with plastic tarvia. The details are shown in Figs. 23(*e*) and 23(*f*).

Bulb beams were selected for the main reinforcement in preference to the 6-in. **I**-beams originally called for, because they had a wider bottom flange for riveting to the beams below and because the small, rounded top flange, permissible in the design on account of the relatively small negative moments, made it possible to work the concrete around them in a satisfactory manner and avoided a flat steel surface near the top of the slab. The rolling of these bulb-beam sections had been discontinued and the rolls destroyed, but arrangements were made to have new rolls made, and it was decided to have the bulb beams for the unpaved center roadway also furnished and erected with their tie-rods. They serve as a protective covering to the otherwise open framing of the central roadway space prior to its being paved.

By arranging the main reinforcement beams longitudinally instead of transversely it was unnecessary to bend them to fit the roadway crown, and the heavy wheel loads were distributed to them with less shear in the concrete.

The lower part of the curbs consist of sections made up of 6-in. **Z**-bars and angles. If the intermediate curbs are removed from their present location these sections will remain, taking the place of bulb-beam reinforcing. This type of bar is also used instead of bulb beams at the middle of the center roadway space for supporting the curbs in case they are moved to that location.

The completed roadway will have a crown of $5\frac{1}{8}$ in., with straight slopes of $\frac{1}{8}$ in. to the foot, connected by a curve at the center, 16 ft. long. Cast-iron drainage scuppers with cast-steel gratings and pipes extending down to the bottom of the stringers are placed at 30-ft. intervals along the outside curbs. If the completed deck is divided into three roadways additional scuppers will be provided at the sides of the central roadway.

The roadway slab is designed for 18 000-lb. wheel loads plus 75% impact, and the unit stresses are comparatively low. The concrete was specified to have a strength of 4 000 lb. per sq. in. after 28 days. Actually, it had an average strength of 4 600 lb. per sq. in. The average weight of the slab is 97 lb. per sq. ft.

Sidewalks.—The sidewalk slabs are 3 in. thick and are reinforced with single mats of $\frac{3}{8}$-in. rods spaced 6 in. transversely and 12 in. longitudinally. They are supported by 8-in., 21-lb. **I**-beams running transversely and spaced 3 ft. 9 in. apart. As previously mentioned, these beams are connected to the fascia girders at their outer ends and the longitudinal beams at their inner ends. The fascia girders are light plate girders, 4 ft. $9\frac{1}{2}$ in. deep, made up

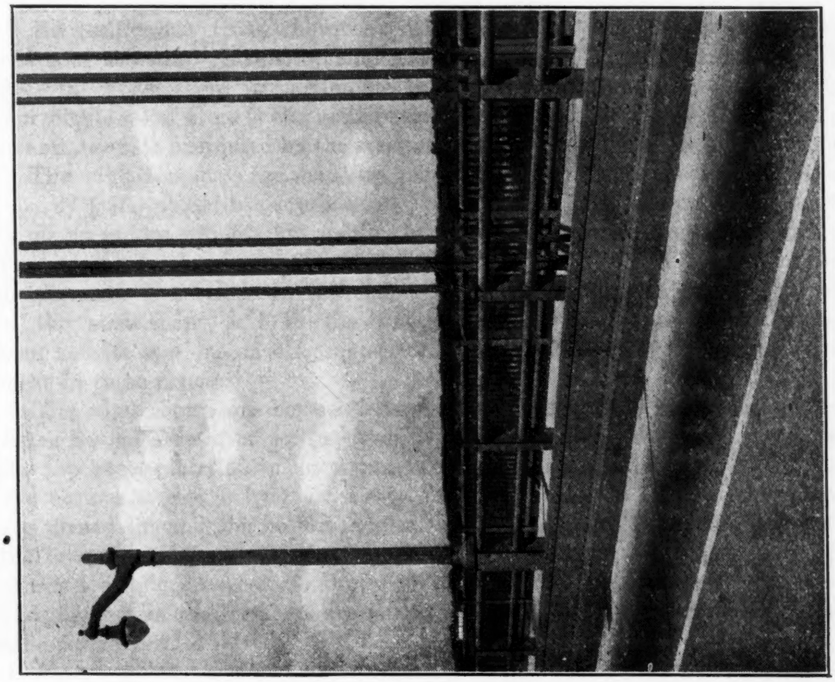


FIG. 25.—VIEW OF DECK, GEORGE WASHINGTON BRIDGE.

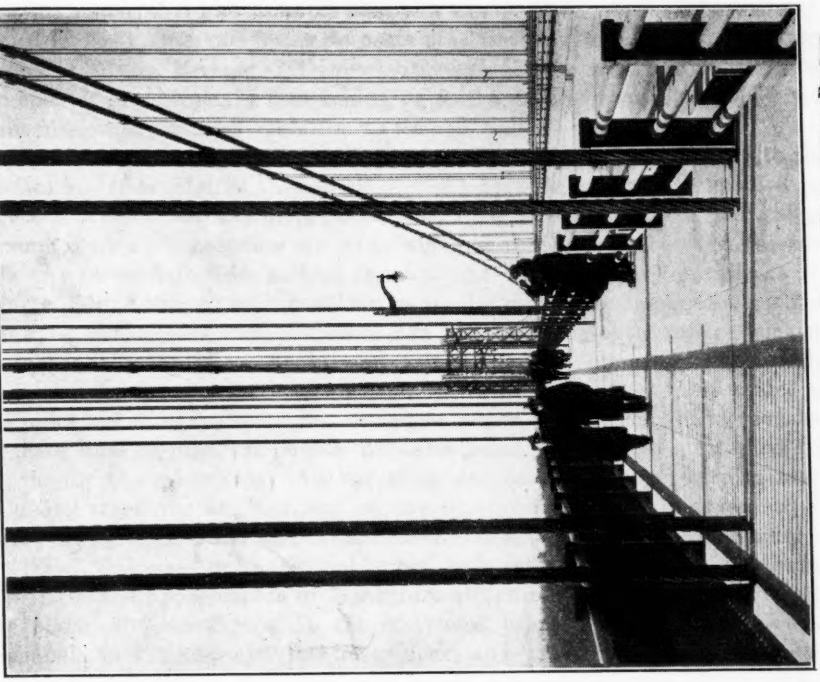


FIG. 26.—VIEW ALONG SIDEWALK, GEORGE WASHINGTON BRIDGE.

of $\frac{3}{8}$-in. material. The longitudinal beams are 14-in., 38-lb. I-beams supported at 20-ft. intervals by angle posts bracketed out from stiffeners on the outside roadway stringer. This was preferred to a direct connection to the ends of the secondary floor-beams, or to the curbs, in order to reduce the transmission of roadway vibration to the sidewalks.

The sidewalks are provided with inner and outer railings connected to the steelwork. (See Fig. 26.) The clear width between railings is 10 ft. 8 in. The suspender ropes, however, pass through the sidewalk slab in two groups of four ropes each, reducing the clear width of the sidewalk between the suspender ropes to 7 ft. 6 in. at each panel point. The posts for the railings are 8-in. 17.5-lb. I-beams, with cast-iron caps, the pipe rails being connected to the webs of the posts with malleable iron fittings. The outer railing consists of top and bottom pipes, $4\frac{1}{2}$ in. and 4 in. in outside diameter, respectively, connected by rectangular steel pickets, $1\frac{3}{4}$ in. by 1 in. in section, welded to the pipes and spaced about 5 in., center to center. The inner railing consists of three lines of pipe, the outside diameter being $4\frac{1}{2}$ in. for the top pipe and $3\frac{1}{2}$ in. for the other two. All the pipes are made of copper-bearing steel. Lighting standards are mounted on the inner railing and are spaced 90 ft. apart, the lights on either side of the roadway being placed opposite each other.

Wind Trusses.—In the initial upper deck construction the wind trusses consist of the upper chords of the future stiffening trusses and diagonals in the plane of these chords. In the completed bridge there will be no wind diagonals in the plane of the lower deck, and the wind load on this deck will be transmitted to the upper deck by the vertical members of the stiffening truss acting with the upper floor-beams as a stiff frame. The lower chords of the stiffening truss, however, will act with the upper chords as chords of the wind trusses, because the diagonals of the stiffening truss will force the lower chord to participate in the stress of the upper chord. Any appreciable relief from such participation, due to the stiffening trusses bowing up and down, is prevented by the resistance of the loaded cable to such motion.

The wind diagonals are made up generally of two carbon-steel angles, 8 by 8 in. by $\frac{5}{8}$ in., placed back to back and held to the under side of the stringers by clamp guides which allow motion between the diagonals and the stringers. The diagonals form an X-system, two panels long, and are connected to alternate floor-beams at the ends and at the center. In the end panels of the side spans and in the four panels at each end of the center span heavier sections of silicon steel are used on account of the greater wind shear in these panels.

The wind chords are box-sections of silicon steel. The upper deck chord, which has an area of 85 sq. in., is made up of two web-plates, 30 in. by $\frac{5}{8}$ in., one top cover-plate, 30 in. by $\frac{1}{2}$ in., two top angles, 6 by 6 in. by $\frac{1}{2}$ in., and two bottom angles, 8 by 8 in. by $\frac{11}{16}$ in. The horizontal legs of the angles are turned in, and the bottom angles are laced together. As now designed, the lower chords are somewhat smaller, having an area of 75 sq. in. The chords are spliced near the end of each panel for the full value of the member and the ends of the members are milled to bear, no additional provision being made for reversal of stress.

The wind trusses in the center and side spans are entirely separate. The center-span truss has a hinged and sliding end connection to each tower on the center line of the bridge, constructed so as to allow angular and longitudinal motion. The side-span trusses have similar hinged and sliding connections at the towers, but the connections at the anchorages are fixed against both motions, the ends of the chords being connected to frames embedded in the anchorage masonry which take the longitudinal reaction, while the wind shear reaction is taken by the anchorage floor-beams.

Analysis of Wind System.—The wind loads acting on the cables and the floor structure are transmitted to the towers and anchorages partly by the cables and partly by the wind trusses. These two carrying members, however, are interdependent since they are connected by the suspenders. In the case of the long center span the wind truss is much more flexible than the cables. Its greater deflection from wind load moves the suspenders from their vertical plane, the lateral component of the suspender pull tending to restrain the deflection of the truss and, at the same time, to increase the lateral deflection of the cables. This restraining effect is particularly great near the middle of the span, where the suspenders are short, and where the lateral component of the suspender pull is greater than the wind load on the floor.

The wind truss, therefore, has not only end reactions at the towers, but reactions all along its length, from the suspenders, which are small near the ends and large near the center. The maximum moment occurs near the quarter-points.

In the case of the relatively short side spans the deflections of the cables and the wind trusses are small, and nearly equal, so that the effect of the suspenders is negligible, and the wind load on the floor was assumed to be carried by the wind truss alone.

To find the moments and shears in the center-span wind truss it was necessary to determine the amount of the lateral suspender pull or reaction at every point along the truss. This was done by a "cut-and-try" method of assuming the lateral suspender pulls, finding the deflections of the cables, suspenders, and truss, and then revising the assumed pulls until these deflections were in agreement at all points; in other words until the deflection of the truss at every point was equal to the sum of the deflections of the cables and suspenders at that point, or (referring to Fig. 27), $d_t = d_c + d_s$; in which, d_t is the lateral deflection of wind truss; d_c, the lateral deflection of cables; and d_s, the lateral deflection of suspenders, at any point.

The deflections of the truss were found by the moment-area method, corrected for the effect of the diagonals, with the floor wind loads acting in one direction and the assumed lateral suspender pulls acting in the opposite direction. The deflections of the cables were found by determining the equilibrium polygon for the wind loads on the cables and the assumed lateral suspender pulls combined, with the horizontal component of the cable pull equal to that due to the vertical loads. The deflections of the suspenders were found by the principle that the deflection of a suspender bears the same relation to its length as the lateral component of its pull bears to its pull, or, referring

to Fig. 27, $\frac{d_s}{h} = \frac{q}{p}$, in which, d_s is the lateral deflection of a suspender; h, the length of the suspender; q, the lateral component of the suspender pull; and p, the suspender pull.

The truss deflections, and the cable deflections with the suspender deflections added, were plotted and the two curves compared. The assumed lateral

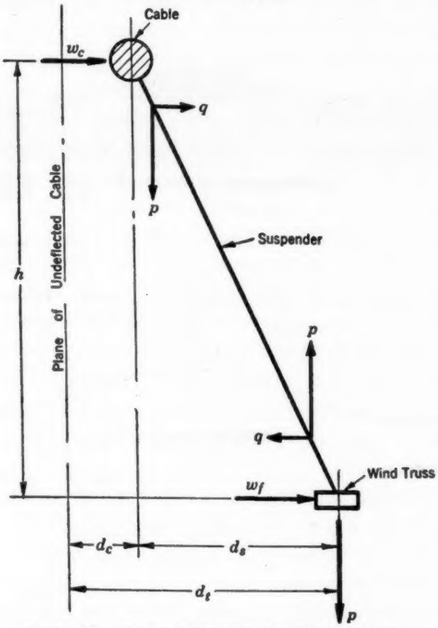


FIG. 27.—DISTRIBUTION OF WIND LOAD.

suspender pulls were then corrected, and the deflections re-calculated until the curves were brought into agreement.

The lateral suspender pulls may be considered as transferring floor wind load to the cables. This distribution of the wind load between the cables and truss is shown in Fig. 28, which also shows the deflection curves and moment and shear diagrams. In this case, which is for the completed bridge, 65% of the floor wind load is transferred to the cables. The maximum truss deflection which occurs at the center is 11.8 ft. A deflection of this magnitude, while not objectionable, is not likely to occur because a steady wind pressure of 30 lb. per sq. ft. over the entire span is highly improbable. To reduce this deflection materially by increasing the chord areas would have been expensive. Even doubling the chord areas would have reduced the deflection to only 10.6 ft.

Deflections and stresses were also determined for the initial upper deck construction; but, as the reduction in wind pressure is greater than the reduction in weight of the structure, the deflections and stresses were found to be smaller than for the completed bridge, the maximum deflection being 10.7 ft.

Stiffening Trusses.—The future stiffening trusses as n w designed have a depth of 29 ft. center to center of chords and are Warren trusses with two

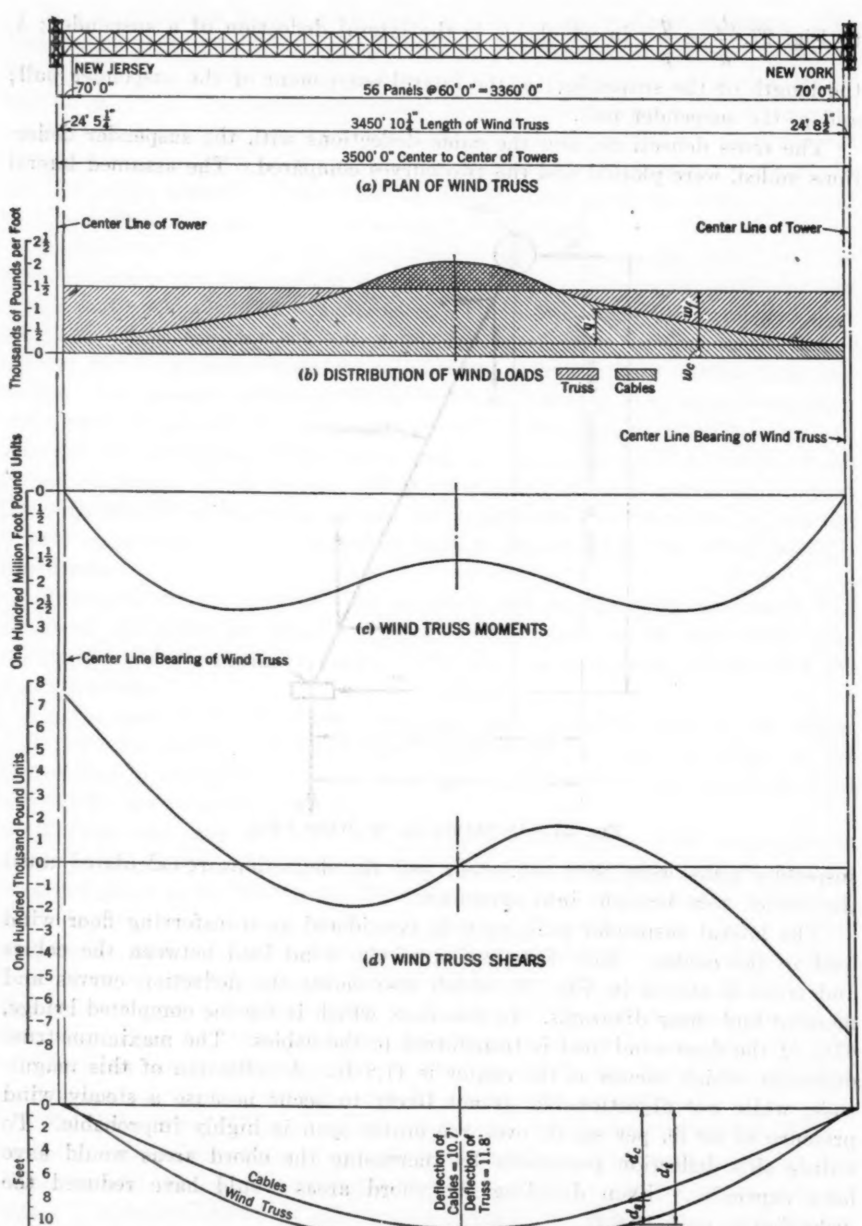


FIG. 28.—WIND SYSTEM, CENTER SPAN.

diagonals in each 60-ft. panel. The make-up of the cross-section of the upper chords, which have an area of 85 sq. in. and are of silicon steel has previously been given in the description of the wind trusses. The lower chords, which have an area of 75 sq. in., are box sections of silicon steel consisting of two web-plates 24 in by $\frac{5}{8}$ in., one top cover-plate, 30 in. by $\frac{1}{2}$ in., two top angles, 6 by 6 in. by $\frac{1}{2}$ in., and two bottom angles, 8 by 6 in. by $\frac{11}{16}$ in. The horizontal legs of the angles are turned in, the long legs of the bottom angles being horizontal and laced together. The diagonals are also box sections, the typical diagonals being of carbon steel and having an area of 39 sq. in. They are made up of two web-plates, 20 in. by $\frac{1}{2}$ in., and four angles, 6 by 4 in. by $\frac{1}{2}$ in., with the 6-in. legs turned in and laced together. Seven diagonals at each end of the center span, three diagonals at the tower ends of the side spans, and four diagonals at the anchorage ends of the side spans, require stronger sections than the typical sections, and these are obtained either by the use of silicon steel, or by heavier sections, or both. Gusset-plates are now provided on the upper chords for the connection of these future diagonals, but are left blank, the holes to be drilled in the field. The center-span stiffening trusses are supported at the towers by pinned hangers connecting the end panel points of the bottom chords to brackets on the towers, as shown subsequently under "Details at Tower," in Fig. 30. Similar hangers support the side-span trusses at the towers, and the shore ends are connected to the frames embedded in the anchorage masonry by the pins now connecting the top chord.

The stiffening trusses are very flexible, having a ratio of depth to span of 1:120 in the case of the center span, and 1:20 and 1:18, respectively, for the New York and New Jersey side spans. As a result they have no appreciable effect upon the deformation of the cables, except for very short and heavy loads, and, even then, their effect is slight. Since the deformation of the cables is so little affected by the truss stiffness and since the trusses are forced to take the same deformation, it follows that the live load unit stresses in the truss chords are practically independent of their cross-sectional area and vary in almost direct proportion to the depth of the truss. The depth used for the trusses is the minimum permitted for the overhead clearance required for the lower deck and the detail arrangement of the floor system. This depth practically determined the live load unit stresses in the chords, and since the unit stresses from wind (as has previously been explained), are only slightly affected by the chord areas, the proportioning of the chords became simply a matter of selecting minimum practical cross-sections and choosing a grade of steel capable of withstanding the imposed unit stresses.

As previously stated, the trusses are designed for a partial live load of 23 000 lb. per ft. of bridge, reduced for length of load and number of loaded lanes, together with the impact percentage as given by the specifications; and, in addition, an extended load of 4 000 lb. per ft. over the entire structure, including the section covered by the partial load. However, it was found in all cases that this load of 4 000 lb. per ft. actually decreased the stresses from the partial load by a very small amount, and it was not used at all in determining the stresses. The lengths of load for the partial live

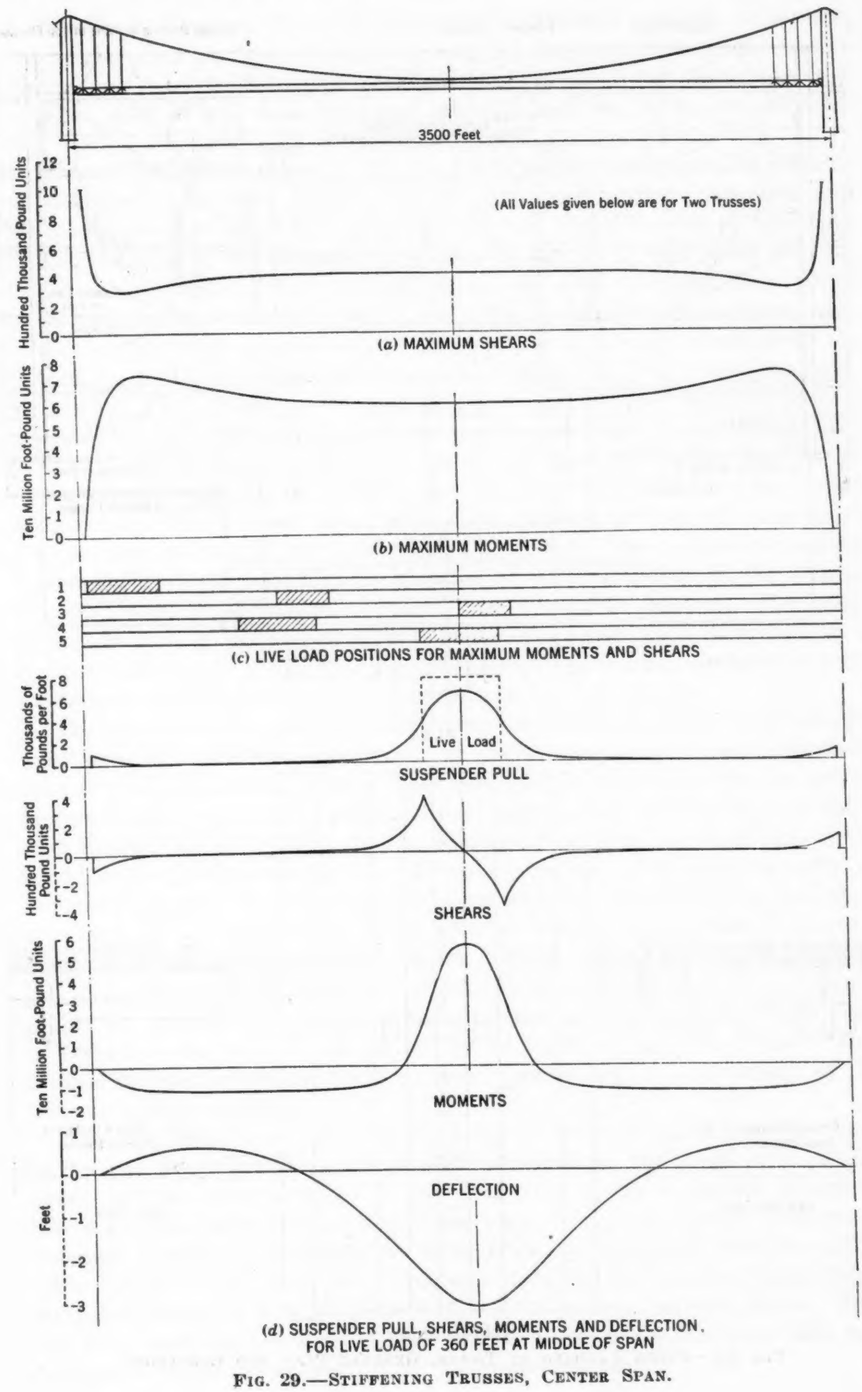
load were taken as multiples of the panel length. By trying various lengths it was found that a loaded length of six panels, or 360 ft., gave maximum chord stresses for the entire center span. For this length the reduced partial load is 7 700 lb. per ft. of bridge and the impact, 10 per cent. For maximum shears in the center span, it was found that the loaded length should be four panels, or 240 ft., the reduced partial live load for this length being 8 900 lb. per ft., with an impact of 13 per cent. For the side spans it was found that a partial live load covering the entire span gave maximum chord stresses, the reduced partial live load being 6 700 lb. per ft., with an impact of 7 per cent. The load positions for maximum shears in the side spans were the same as those for a simple span.

In the center span the maximum unit stresses from live load and impact are 15 800 lb. per sq. in. compression on the gross area of the upper chord, and 22 500 lb. per sq. in. tension on the net area of the lower chord. These maximum stresses occur near the ends of the span. The maximum unit stresses from wind are 15 500 lb. per sq. in. compression in the upper chord and 19 100 lb. per sq. in. tension in the lower chord. These maximum stresses occur near the quarter-points of the span. By combining the maximum live load and impact stresses with one-half the wind stresses at the same point in the truss, or the maximum wind stresses with one-half the corresponding live load and impact stresses, as required by the specifications, the total stresses (including the small stresses due to temperature) were found to come within the specified allowable unit stresses for silicon steel for such load combinations. These specified stresses are: A compression of 24 400 lb. per sq. in. for the upper chord $\left(\frac{l}{r} = 60.5\right)$; and a tension of 29 700 lb. per sq. in. for the lower chord.

In the side spans the unit stresses from live load and impact are somewhat less than in the center span, and the wind stresses very much less, so that the total stresses are well within the allowable unit stresses for silicon steel, although greater than those for carbon steel.

In the final calculations for the live load moments and shears in the stiffening trusses, the formulas of the so-called "exact" or "deflection" theory were used for most of the center span. For the side spans, however, and for short sections at each end of the center span, these formulas give incorrect results because they neglect the stretch of the long suspenders and because the supports for the cables and trusses are not in the same vertical line. For the side spans and for the ends of the center span, therefore, a "cut-and-try" method was used, which consisted in assuming the distribution of live and dead load between the truss and cables, and then computing the deflections of the truss and the deflections of the cables, including the stretch of the suspenders. The assumed load distribution was then revised until these two sets of deflections were in agreement.

Fig. 29 gives the curves of maximum moments and shears for the center span and also the curves of loading, shear, moment, and deflection for a typical load case.



(*d*) SUSPENDER PULL, SHEARS, MOMENTS AND DEFLECTION, FOR LIVE LOAD OF 360 FEET AT MIDDLE OF SPAN

FIG. 29.—STIFFENING TRUSSES, CENTER SPAN.

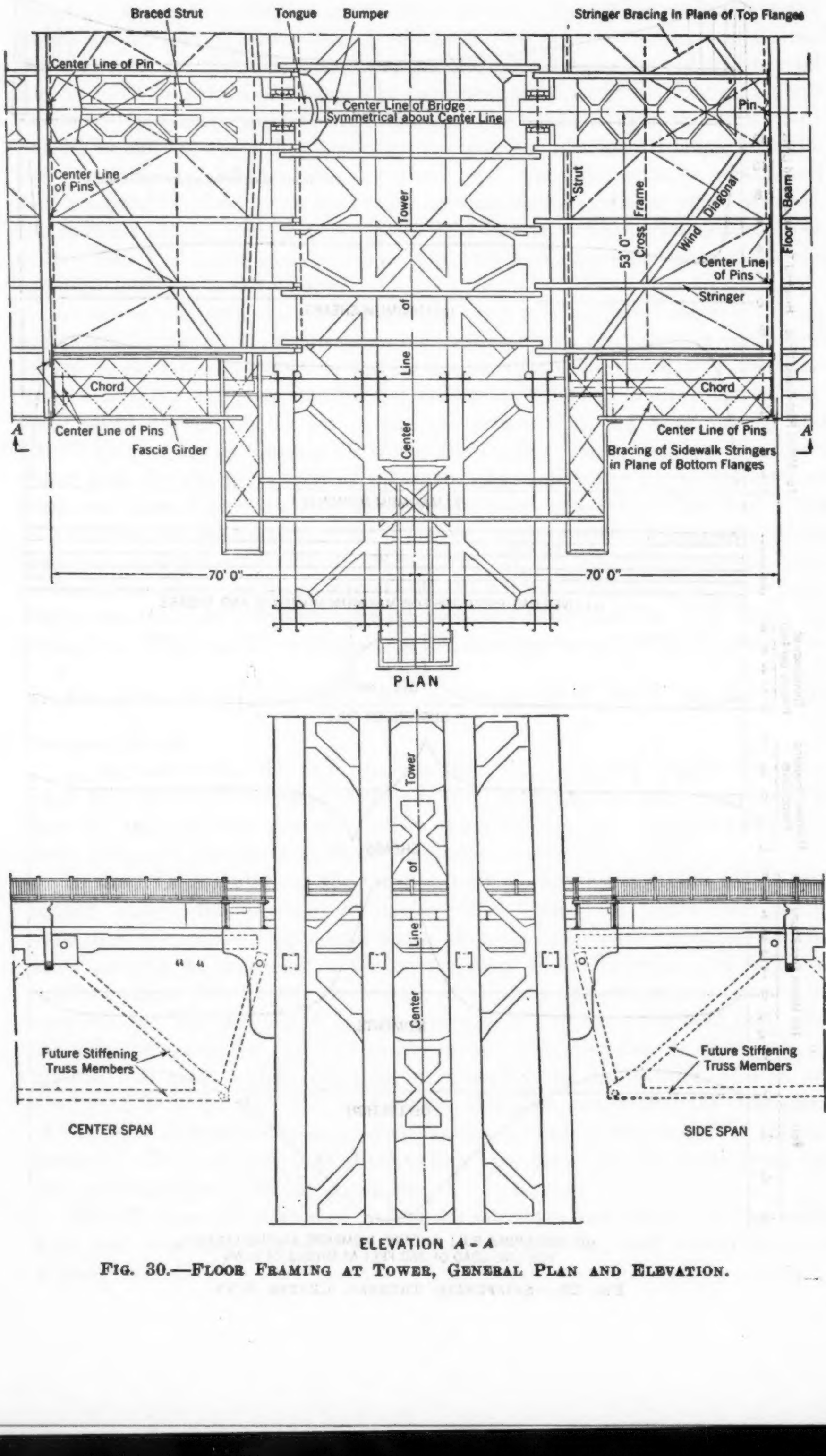


FIG. 30.—FLOOR FRAMING AT TOWER, GENERAL PLAN AND ELEVATION.

Details at Towers.—The connections that transmit vertical, transverse, and longitudinal reactions from the suspended floor structure to the towers are designed to permit unrestricted angular movement in both horizontal and vertical planes as well as longitudinal movement. Figs. 30, 31, and 32 show the details at the towers for both the center and the side spans.

At each tower the steelwork of the roadway is supported on the transverse struts of the tower. The stringers which span between the tower and the first suspended floor-beam are pin-connected to the floor-beam, and have sliding bearings on the tower strut. The wind chords in this panel are connected at the floor-beam by horizontal pins to the wind chords of the adjacent panel, and the ends at the tower are supported by brackets from the outside stringers. The wind laterals in this panel consist of (1) diagonals from the ends of the chords to the center of the floor-beam, where a transverse shear connection and a horizontal pin-joint are provided; and (2) transverse struts from the ends of the chords to a connection with the tower at the center line of the bridge. This connection consists of a tongue, 5 ft. wide, made up of plates and stiffener angles, which is free to move longitudinally over the top of the tower strut between cast-steel guides. The outer faces of these guide castings are cylindrical and bear against the concave faces of castings fixed to the tower strut, thus allowing rotation as well as sliding of the tongue, the wind shear being transmitted to the tower by these castings. The longitudinal motion of the tongue at each end of the center span is limited by a bumper fastened to a longitudinal tower strut, the contact surface between the tongue and the bumper being curved to allow for rotation.

The clearance between the tongue and the bumper is made large enough to allow some longitudinal movement of the entire suspended floor structure, even at the highest temperature. Such longitudinal movement may be caused by longitudinal wind and breaking forces, these forces amounting to a total of 3 500 000 lb. If resisted by the suspenders and cables alone this force would move the structure 24 in. The bumpers were introduced in order to reduce the length of the expansion details and, in limiting the motion, they take part of the longitudinal force. At the highest temperature the movement permitted by the bumpers is $4\frac{3}{4}$ in., and the bumper reaction is 2 800 000 lb. This force is transmitted to the bumper by a longitudinal strut between the tongue and the intersection of the diagonals at the floor-beam. This strut is braced to the bottom flanges of the adjacent stringers to form a wide trussed member which carries the moment caused by the eccentricity of the transverse wind reaction.

The expansion joint in the roadway slab on the center-span side of the tower is formed by two wide gratings, one attached to the floor steel of the tower and the other attached to the floor steel of the suspended structure. These gratings resemble combs in form, with their teeth intermeshing over a common support upon which the teeth from the suspended structure slide. (See Fig. 33.) The teeth are rolled steel flats, 56 in. long and $\frac{15}{16}$ in. thick, welded together in 2-ft. sections, with $1\frac{1}{16}$-in. spacers between them. They are 5 in. deep on the moving side and $6\frac{1}{4}$ in. deep on the fixed side, and their top surfaces are roughened by transverse grooves.

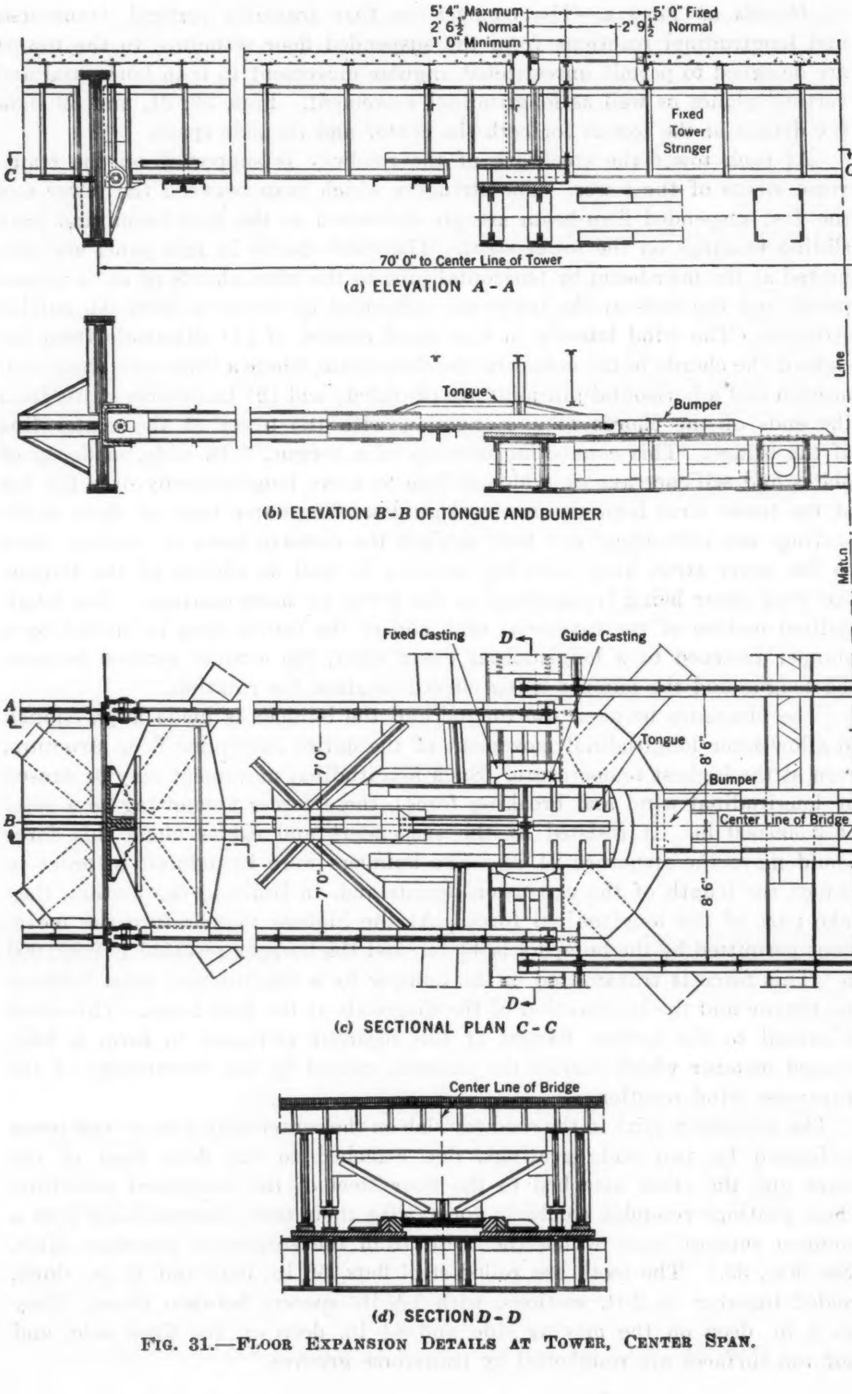


FIG. 31.—FLOOR EXPANSION DETAILS AT TOWER, CENTER SPAN.

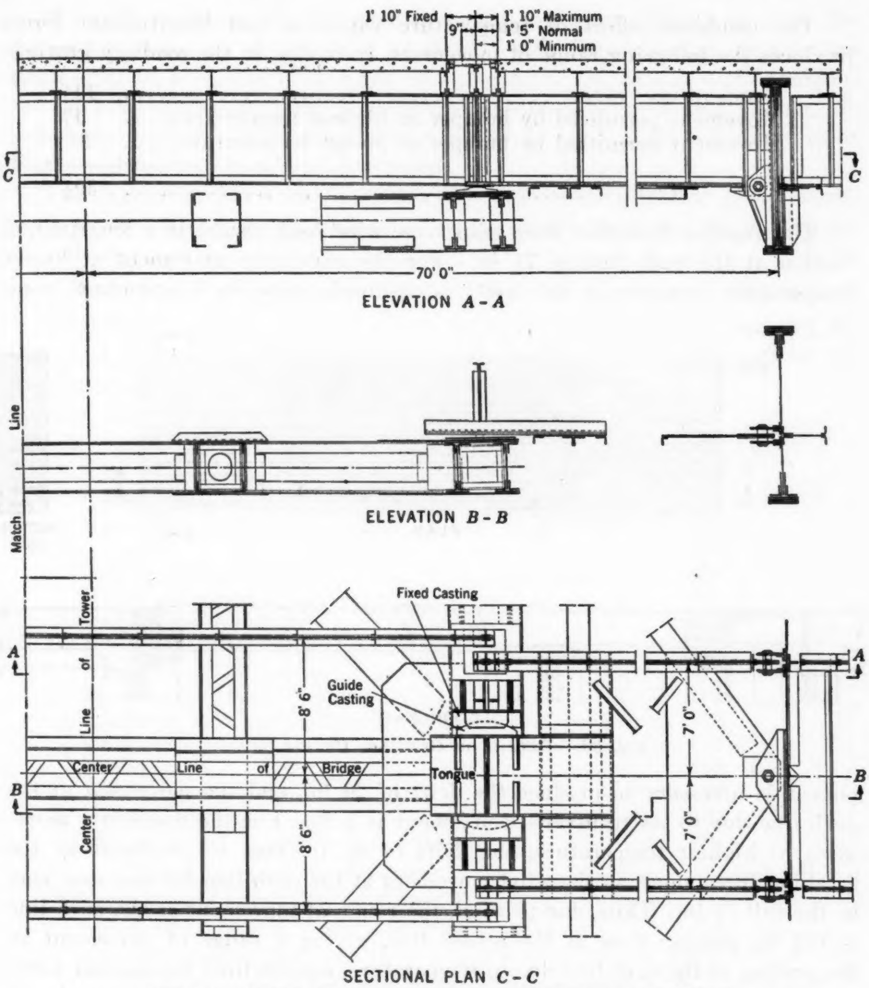


FIG. 32.—FLOOR EXPANSION DETAILS AT TOWER, SIDE SPAN.

The temperature range of 110° produces the following range of movement, in inches, in the center-span gratings at each tower:

Change in length of floor..............................	$= + 15$
Deflection of tower at grating level................	$= + \frac{5}{8}$
Change in length of floor due to change in camber.....	$= + \frac{5}{8}$
Angular tilt of end of stiffening truss...............	$= - 1\frac{3}{4}$
Total..	$= + 14\frac{1}{2}$

The variation in bumper clearance due to temperature is the same as that in the grating, with the exception of the effect of the angular tilt of the stiffening truss which is $\frac{3}{4}$ in. less at the bumper level. Thus, the total change in bumper clearance due to temperature is $15\frac{1}{4}$ in., and it varies from a minimum of $4\frac{3}{4}$ in. at + 105° to a maximum of 20 in. at − 5 degrees.

The combined effect of temperature variation and longitudinal forces produces the following range of movement, in inches, in the roadway grating:

Temperature range	$14\frac{1}{2}$
Movement permitted by bumper at highest temperature	$4\frac{3}{4}$
Movement permitted by bumper at lowest temperature	20
Total	$39\frac{1}{4}$

The angular deflection from transverse wind load results in a longitudinal motion at the curb line of $7\frac{1}{2}$ in. For the maximum movement at lowest temperature, however, a 45° wind is assumed, since its longitudinal com-

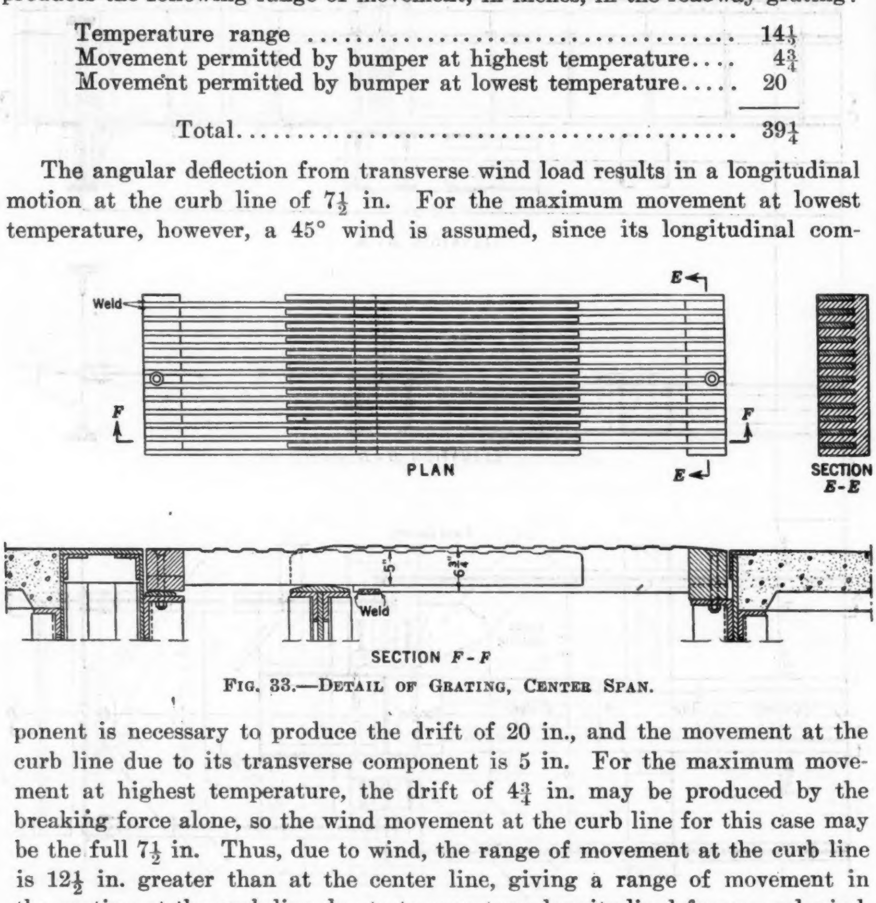


Fig. 33.—Detail of Grating, Center Span.

ponent is necessary to produce the drift of 20 in., and the movement at the curb line due to its transverse component is 5 in. For the maximum movement at highest temperature, the drift of $4\frac{3}{4}$ in. may be produced by the breaking force alone, so the wind movement at the curb line for this case may be the full $7\frac{1}{2}$ in. Thus, due to wind, the range of movement at the curb line is $12\frac{1}{2}$ in. greater than at the center line, giving a range of movement in the grating at the curb line due to temperature, longitudinal forces, and wind, of $51\frac{3}{4}$ in., of which, $19\frac{1}{2}$ in. is a closing from normal and $32\frac{1}{4}$ in. is an opening from normal.

A partial live load at one end of the span causes an angular tilt of the stiffening truss outward from the tower at the loaded end, which causes an opening at the grating of $3\frac{3}{8}$ in. However, at the unloaded end of the truss, there is an inward angular tilt which reduces the possible longitudinal movement permitted by the bumper by $\frac{3}{8}$ in., so that the net opening from live load is 3 in. The simultaneous occurrence of this partial live load with maximum wind, temperature, and longitudinal force, is so improbable that it was not considered necessary to allow for a combination of all these movements in the expansion joint, and the grating was designed to allow a range of movement of 52 in., of which $18\frac{1}{2}$ in. is a closing from normal and $33\frac{1}{2}$ in. is an opening from normal.

For the side spans the connections to the tower are similar to those for the center span, but the motions provided for in the expansion grating are much smaller, having a total range of only 10 in. Since longitudinal forces on the side spans are taken into the anchorages by embedded frames, no bumper is provided at the tower, and there is no need to allow for movement from longitudinal forces in the grating.

The expansion joints in the sidewalks are formed by sliding plates allowing a range of movement of 62 in. for the center span, and 16 in. for the side spans.

Weight of Structure and Principal Quantities for Main Bridge

Dead Load.—The weight of cables and suspenders is not uniform, and, for convenience in design, equivalent uniform loads which would produce the same moment at the middle of the span were computed.

The make-up of these loads for the center span is, as follows:

	Equivalent dead loads, in pounds per foot of center span:	
Present Structure:		
Cables, including wrapping and hand ropes	11 120	
Suspenders, including cable bands and other details	620	
Top chords	830	
Wind diagonals	185	
Main floor-beams	2 080	
Stringers and bracing	2 085	
Secondary floor-beams	835	
Bulb beams and tie-rods	1 075	
Sidewalk steel, including fascia girders	530	
Curbs	495	
Railings	260	
Side roadway slabs (2 @ 28 ft. 9 in.)	4 850	
Sidewalk slabs	950	
Electrical equipment	120	
Total for present structure		26 035
Additions for Completion of Upper Deck:		
Alterations to inside curbs and railings	—75	
Center roadway slab (30 ft. 6 in)	2 610	
Total additions for completion of upper deck		2 535
Total for completed upper deck		28 570
Lower Deck:		
Bottom chords, diagonals, and verticals	1 570	
Main floor-beams	1 200	
Longitudinal girders	1 210	
Lateral bracing	100	
Intermediate floor-beams	500	
Stringers and bracing	620	
Braking trusses	60	
Rails, ties, fastenings, etc	2 100	
Total for lower deck		7 360
Total for completed bridge		35 930
Allowance for possible additions		3 070
Total design load: Center span		39 000

The corresponding dead load for the side spans is 40 000 lb. per ft., the greater weight being due to the steeper slope of the cables, the longer suspenders, and the heavier cable bands. For calculating the dead load cable polygon (which was required for determining the length of suspenders and indirectly for setting the strands to the proper sag during the erection of the cable), the actual panel load concentrations were used instead of uniform loads.

Principal Quantities for Bridge Proper.—As built to date (1932), the George Washington Bridge contains the following quantities of steel, in pounds:

Steel cable wire and suspender rope		60 000 000
Silicon Steel:		
Towers	47 200 000	
Floor	16 200 000	
Total		63 400 000
Carbon Steel:		
Towers	34 600 000	
Floor	24 000 000	
Anchorages	8 500 000	
Total		67 100 000
Heat-treated eye-bars, pins, and bolts		10 200 000
Cast steel and cast iron		5 300 000
Total steel in present structure		206 000 000
Estimated quantity of steel, in pounds, in the future lower deck		25 000 000

The quantities of concrete and granite facing, in cubic yards, are:

New York anchorage	114 300
New Jersey anchorage	25 300
New York tower base	11 500
New Jersey tower base	38 000
Roadway and sidewalk slabs on suspended structure	6 700
Total concrete in present structure	195 800
Estimated granite and concrete to complete New York anchorage	27 000
Estimated concrete to finish roadway pavement	3 000

Design Organization

The design work for the George Washington Bridge was carried out under the direction of the writers of this paper and Willener A. Cuenot, M. Am. Soc. C. E., Chief Draftsman.

Acknowledgment

The writers wish to acknowledge the valuable assistance of Douglas A. Nettleton, Jun. Am. Soc. C. E., a member of the staff, in the preparation of this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

GEORGE WASHINGTON BRIDGE: DESIGN OF THE TOWERS

By Leon S. Moisseiff,[1] M. Am. Soc. C. E.

Synopsis

The unusual type of tower adopted in the design of the George Washington Bridge is discussed in this paper. The entire load of the bridge is applied at the top of the inner columns of the tower and is distributed between all columns as the stress proceeds downward, until finally the reaction on the masonry is practically uniform. The paper demonstrates how this tower was analyzed; methods used to test, analytically, the efficiency of the design are related; stress-strain measurements on a celluloid model of a tower bent are described; and results are given of stress-strain measurements on the completed towers, which show the distribution of the load in the tower columns. These final measurements demonstrate the agreement between engineering theory and the actual behavior of the structure. Tests of large sized columns were made in conjunction with the National Bureau of Standards, and their results are reported. Conclusions are drawn as to the efficiency and safety of structures of this magnitude.

Introduction

By their position in the landscape, their rising mass, and the emphasis which the ascending curves of the cables throw to them, the towers of a suspension bridge exert a preponderant influence on the æsthetic impression of the observer. When the bridge is viewed as a whole they at once attract the eye and concentrate the attention. Of the three principal parts of a suspension bridge—the anchorages, cables, and towers—the anchorages are too far away and are too often hidden by the design or by other structures to be felt as part of the picture, while the cables by their continuity and relatively small size do not present a restful object. As a matter of fact,

Note.—Discussion on this paper will be closed in March, 1933, *Proceedings*.

[1] Advisory Engr. on Design, The Port of New York Authority; Cons. Engr., New York, N. Y.

the towers of a suspension bridge not only sustain the entire weight of the superstructure, but also the upward pull of the cables at the anchorages, and the greatest forces are concentrated on them. The statical importance of the towers is justly felt by the public at sight, and they, therefore, demand the best æsthetic treatment.

Large public works have their history and origin that influence their development and final form. It requires time for the demand for a public bridge involving large expenditures and engineering difficulties to grow and to become strong enough to be realized. In the course of time the need for traffic facilities grows and the demand for them becomes more pressing. At the same time, the available funds, the engineering knowledge, the technical capacities of the mill, the fabricator, and the erector, also grow. During the period in which the idea of the bridge is being "incubated," the engineering phases of the structure take form through public opinion and engineering study, which mature later into the completed structure.

The story of the growth of the idea for bridging the Hudson River, which has finally culminated in the George Washington Bridge, has been completely told in the paper[2] by O. H. Ammann, M. Am. Soc. C. E. It began in the last ten years of the Nineteenth Century and continued through forty years. The great advances made during this period in the theory of structures, the knowledge of materials, the making of steel, the fabrication and erection of bridges, all find themselves reflected and embodied in the bridge as built; but so also do many notions and traditions of older times. This especially applies to the towers of the George Washington Bridge.

The age of metal structures is still young; that of stone goes back to time immemorial. For many centuries, habit, which determines æsthetic feeling to the largest extent, saw beauty in the massiveness of stone only. The shadow of the well-proportioned masonry towers of the old Brooklyn Bridge fell upon the proposed towers of the George Washington Bridge.

The genetics of the bridge project over the Hudson River were such that, conforming to public opinion and its habitual taste, the bridge was first pictured with massive towers. When after years the project was finally approaching realization, the towers were built as steel structures of strength and stability to sustain all the loads that may come on the bridge, and provisions were made in their design to allow for embedding or enclosing them in concrete masonry so as to obtain the effect of a monumental structure. As a matter of fact, the appearance of the bare steel towers is better than was expected, and they seem to have gained the favor of the general public. The writer is of the opinion that they will probably remain without enclosure, as shown in Fig. 1.

The fact that the towers were conceived to be ultimately encased in masonry is important to the understanding of their design and their articulation. It determined the main features and the novelty of proportioning. The problem to be met was how best to design steel towers for the required dimensions and forces.

[2] *Proceedings*, Am. Soc. C. E., August, 1932, p. 969.

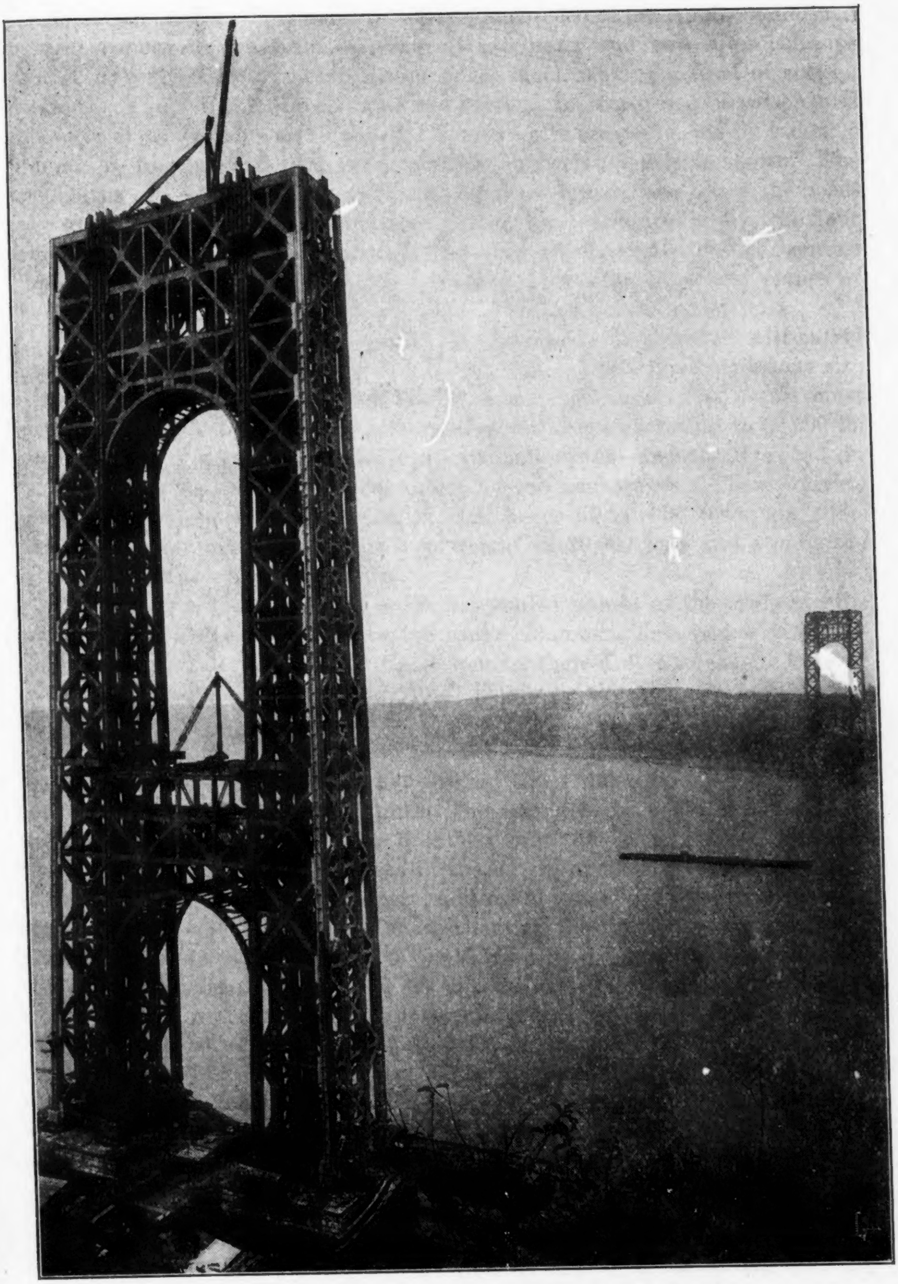

FIG. 1.—VIEW OF NEW JERSEY TOWER DURING CONSTRUCTION.

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As shown in Fig. 2(b) each tower is composed of four rigidly connected upright frames. Each frame consists of two inner and two outer columns (Fig. 2(a)) with bracing between to connect them, making a total of sixteen columns. The inner eight columns are vertical in transverse elevation and the outer eight columns are inclined. The two sets are connected by a double bracing of diagonal and horizontal members in twelve panels as shown. The two halves of each frame are connected by three braces, one below the roadway and the other two close together at the top. Longitudinally, the four frames are held together by a double bracing of diagonals and horizontals in the planes of the inner and outer columns as well as in several planes of the braces to assure unity of action and equal distribution under load.

The steel towers are proportioned to sustain the entire load ultimately imposed on them. To reach a conception of these forces the tremendous size of the George Washington Bridge should be fully realized. The towers must sustain the weight of the entire suspended structure—amounting to 39 000 lb. per ft. in the center span, 41 400 lb. in the side spans—and the live load on the bridge, as well as the vertical uplift on the anchorages. These weights produce a vertical reaction on top of each tower of 112 000 tons. In addition, each tower must sustain its own weight of 20 000 tons and almost the full wind pressure on the structure.

The saddles transmitting the entire suspended weight of the bridge to the towers are centrally located above the inner columns. This makes it impossible to engage all members of the tower to their full efficiency. Part of the steel is required to transmit and distribute the stress to all sixteen columns.

This condition could have been avoided by transferring one-half the load to the outer columns directly by placing the saddles and cables midway between the inner and outer columns. The arrangement would have increased the distance between center lines of cables from 106 ft. to 144 ft. It would have caused an equivalent increase in the length of all floor-beams, resulting in an additional 12 000 tons of steel. Adding to it the increased wire required to sustain this additional weight and estimating the total increase at contract prices, an extra cost of \$2 800 000 would have been incurred. The effect on the anchorages has been omitted in this estimate; it would add still more to the cost. On the other hand, to locate the saddles and cables concentrically would have saved less than \$100 000 in the steel of the towers. The advantage of this arrangement would be the central location of the cable reaction in the tower bent and concentric distribution of stress that is assumed to follow from it. Its disadvantage is this very considerable additional cost. The more economical design was adopted.

Analysis of Steel Towers

In the computations and the proportioning of the members, the four frames forming the tower were assumed to participate equally in sustaining all vertical loads. Therefore, one frame only was analyzed for loading conditions, taking one-quarter of the total load. Each unit forms a three-story frame with double diagonal bracing between the columns. The frame is statically

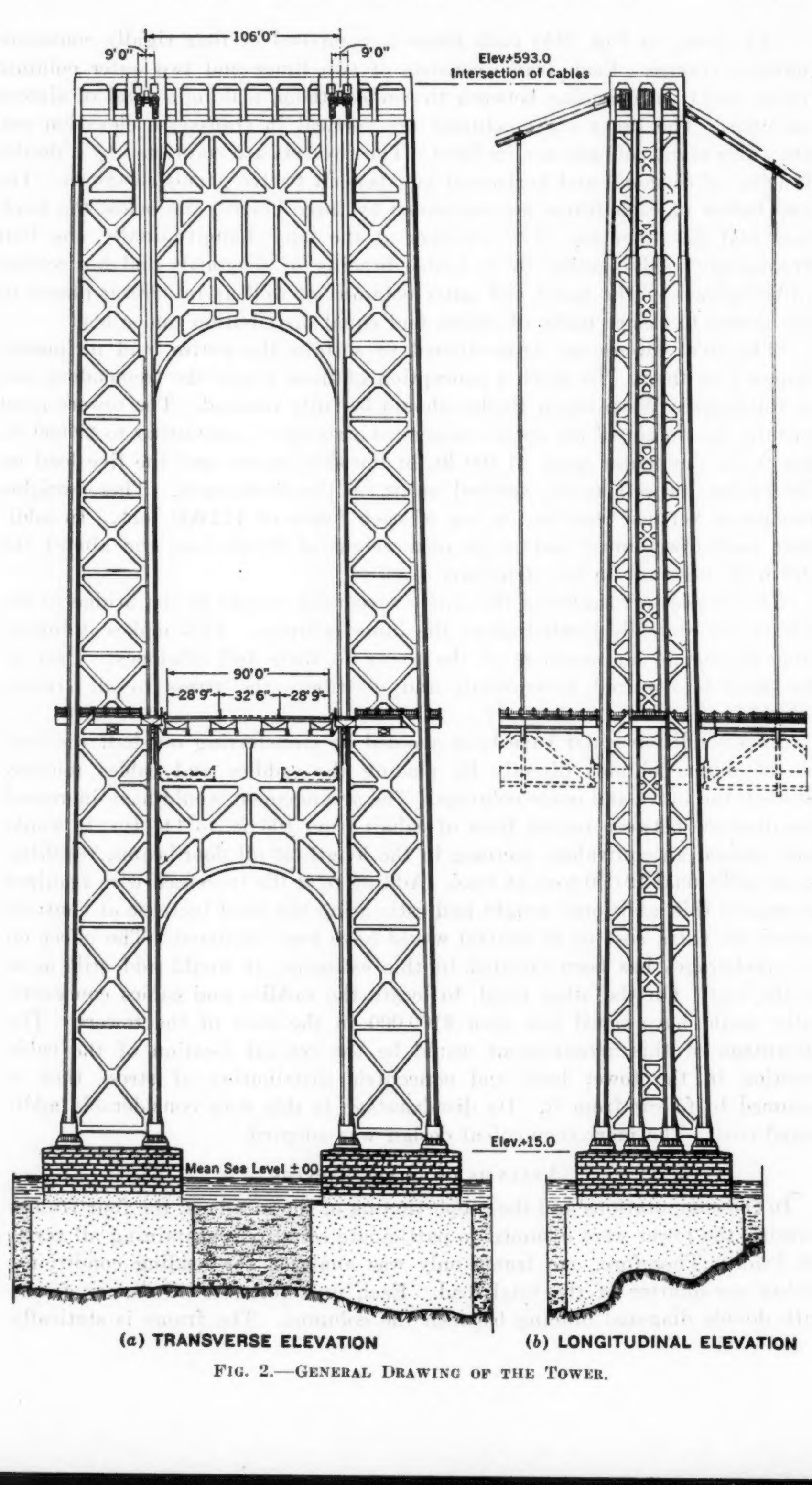


FIG. 2.—GENERAL DRAWING OF THE TOWER.

indeterminate to a high degree, but its elastic behavior is easily visualized. The main external loads are the cable reactions applied at the center of the inner columns. These loads compress the inner columns and tend to shorten them. The braces prevent the columns from bending and part of the stress is forced through the diagonals to the outer columns.

To facilitate the computation of the stresses, the frame was first divided into two superimposed parts, each containing a single diagonal system between the columns. The frame was then transformed into a statically determinate system by passing a vertical plane along the center line, and each member cut by this plane was replaced by an unknown force. This, however, was not quite sufficient to transform the frame into a statically determinate system, some additional redundant members of the braces had to be cut, thus increasing the number of unknown forces to be determined by the applications of the elastic theory. As seen in Fig. 3, a total of sixteen unknowns is required

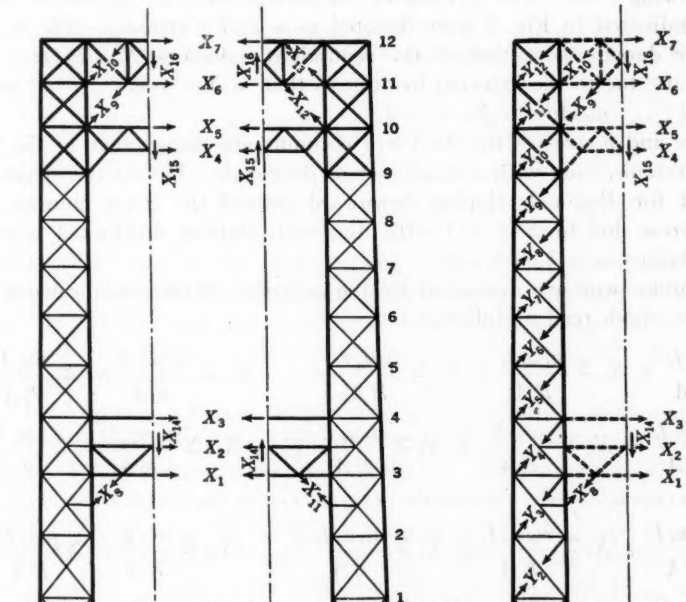


FIG. 3.—FORCE DIAGRAM; DESIGN OF TOWERS.

to make the structure statically determinate for any loading case. Each half of the frame acts as a vertical cantilever, subject to the external and unknown forces. It will be readily seen that for symmetrical vertical loads the forces, X_{14}, X_{15}, and X_{16}, will be zero, and that also Force X_{11} = Force X_8; Force X_{12} = Force X_9; and Force X_{13} = Force X_{10}. For this case of loading, therefore, the number of unknown forces is reduced to ten.

Horizontal forces acting at corresponding points on each side of the center line in the same direction and of equal magnitude will result in the forces, $X_1, X_2, X_3, X_4, X_5, X_6$, and X_7, being equal to zero, and Force X_8 = Force X_{11}; Force X_9 = Force X_{12}; and Force X_{10} = Force X_{13}, thus reducing the number of unknowns to six.

Following this procedure the elastic theory furnishes the number of equations necessary to compute the numerical values of the unknown forces for any loading. These are then treated as external loads acting on a statically determinate system. The application of this theory is well known. At the outset it requires the assumption of the sectional areas of all members of the tower. The computations are repeated several times, therefore, until a close agreement is reached betwen the assumptions made and the results obtained.

The analysis and the procedure for establishing the equations was along the following lines: The stresses in the system with all redundant members cut as indicated in Fig. 3 were denoted as u and p stresses. The u stresses are those due to the action of the redundant members, X_1, X_2, etc. The p stresses are due to the external loading so that u_1 due to $X_1 = -1$; u_2 due to $X_2 = -1$; ... u_{16} due to $X_{16} = -1$.

The u and p stresses for the tower columns are the average of the stresses of two systems, each with a single set of diagonals. If v is the stress due to $X = -1$ for diagonals sloping downward toward the inner column, and w is the stress due to $X = -1$ with diagonals sloping downward toward the outer columns, $u = \frac{1}{2}(v + w)$.

The unknowns are evaluated by the solution of the simultaneous elastic equations which read as follows:

$$X_1 \Sigma \frac{u_1^2 L}{EA} + X_2 \Sigma \frac{u_1 u_2 L}{EA} + X_3 \Sigma \frac{u_1 u_3 L}{EA} + \ldots X_{16} \Sigma \frac{u_1 u_{16} L}{EA} = \Sigma \frac{p u_1 L}{EA} \quad \ldots (1)$$

$$X_1 \Sigma \frac{u_2 u_1 L}{EA} + X_2 \Sigma \frac{u_2^2 L}{EA} + X_3 \Sigma \frac{u_2 u_3 L}{EA} + \ldots X_{16} \Sigma \frac{u_2 u_{16} L}{EA} = \Sigma \frac{p u_2 L}{EA} \quad \ldots (2)$$

....

$$X_1 \Sigma \frac{u_{16} u_1 L}{EA} + X_2 \Sigma \frac{u_{16} u_2 L}{EA} + X_3 \Sigma \frac{u_{16} u_3 L}{EA} + \ldots X_{16} \Sigma \frac{u_{16}^2 L}{EA} = \Sigma \frac{p u_{16} L}{EA} \quad \ldots (16)$$

in which, Equations (3) to (15), inclusive, can be interpolated and in which, L = length of members; A = area of members; and E = modulus of elasticity, which being constant eliminates itself.

The stress in any member of the bracing between the bents is equal to,

$$s = p - X_1 u_1 - X_2 u_2 - X_3 u_3 \ldots X_{16} u_{16} \ldots\ldots\ldots\ldots (17)$$

The fact that two systems—each with a single set of diagonals between the columns—have been superimposed neglects the effect of participation stresses caused by the double diagonal bracing. A vertical load on the towers causes shortening in the columns and thereby compresses the diagonals attached to them. Such stresses, of an inductive nature, are called participation stresses. These stresses are purely of local character and do not in any way

affect the unknowns previously mentioned and, hence, the relative distribution of stresses in the columns remains the same. The participation stresses affect mostly the diagonals and the horizontals, the effect on the column stresses being very small. The stresses in the diagonals could be approximated very closely in a simple manner for each panel independently after the column stresses have been evaluated. This was done in fact in the earlier computations until close agreement was reached between the assumed and required areas. In the final computations, however, one set of diagonals was introduced as unknowns and their values were found, together with the stresses in the other members of the bent, by application of the elastic theory similar to the first part of the analysis. The previously computed unknowns were introduced as external forces. The arrangement of the unknowns for participation stresses is shown in Fig. 3(*b*). The analysis was as follows: Denote by $\overline{u}_1$ stress due to $Y_1 = -1$; $\overline{u}_2$ stress due to $Y_2 = -1$; ... $\overline{u}_{12}$ stress due to $Y_{12} = -1$.

Let $\overline{p}$ denote stress due to external loading plus stress due to the forces, X, as previously found, so that:

$$\overline{p} = p' - X_1 v_1 - X_2 v_2 \ldots X_{16} v_{16}$$

After solving the elastic equations, the stress becomes:

$$s = \overline{p} - Y_1 \overline{u}_1 - Y_2 \overline{u}_2 \ldots Y_{12} \overline{u}_{12}$$

It is assumed also in this more refined analysis that the horizontal shear between the inner and the outer columns is equally sustained by both members of the double diagonal bracing. To eliminate this assumption all unknowns of both systems should be introduced simultaneously, but this was not considered necessary.

The loading case of most importance is the vertical cable reaction acting on the center line of the inner columns. The three braces between the bents keep the columns in vertical position. The tendency of the bents would be to deflect inward at the top, but this is prevented by the top brace. They tend to curve outward, therefore, through their height and cause tension in the middle and lower brace. The compressive force in the top brace amounts to 5.8% of the total cable reaction as compared to a tension in the middle and lower brace of 3.9% and 1.3%, respectively.

The reaction of the saddles on the inner columns is distributed gradually by their elastic shortening through the diagonals to the outer columns. The stress thus transmitted is small at the top and gradually increases toward the bottom. The outer columns in the top panel sustain only 4.3% of the reaction, which increases to 37.8% in Panel 5-6 and reaches 46.5% at the bottom. The diagonals in the upper panels are most effective in transferring stress to the outer columns.

Longitudinal Tower System

The stretching and contracting of the cables in the side spans caused by live load and temperature changes will bend the towers and produce stresses

in them. It is evident that the towers will follow the tremendous forces of the cables and that their tops will be pulled into a position such that the equilibrium of forces is again established. This will be resisted by the posts as well as the diagonals in both longitudinal and transverse planes. The tower will act as a unit, each frame contributing in proportion to its distance from the neutral axis. Two distinct steps are required to analyze the stresses caused by the bending of the towers. One is to find the horizontal force at the tops of the inner columns which, together with the vertical cable reaction and the tower weight, will force the structure to deflect in accordance with the cable movements; and the second is to compute the stresses in the towers, which act in this condition as vertical cantilevers.

Longitudinal wind forces acting directly on the towers and wind forces transmitted from the suspended structure will also tend to deflect the towers. For this case, however, the towers are held in place by the cables and act as vertical beams fixed at the base and hinged at the top.

In computing the bending stresses due to longitudinal bending and wind, the two inner frames were assumed to be replaced by similar structures acting in the plane of the outer frames. The sections of the inner frames were reduced according to their distances from the center of gravity of the acting masses. The diagonals in the plane of the frames were also taken into account in resisting distortion.

Each tower actually consists of four vertical trusses in longitudinal planes. These trusses are tied together by four horizontal braces at Panel Points 4, 10, 11, and 12. The horizontal force at the top in the case of tower bending, the reaction of the saddles, and the transferred wind at floor level in the case of longitudinal wind pressure are all applied directly on the two trusses formed by the inner columns only. The two remaining trusses formed by the outer columns are forced into action by the horizontal braces. If the tower is treated for symmetrical loads, it is four times statically indeterminate for bending, the shears transferred through the four horizontal braces from the inner to the outer columns being the unknowns. For longitudinal wind pressure on the tower the unknowns are increased to five, the fifth being the wind reaction at the top.

The horizontal braces have been found to be very efficient. The stress transferred to the outer columns by bending is equal to 43% in Panel 5-6 and increases to 47% at the base; the wind stresses are equally well distributed.

Studies of Efficiency of Tower Design

The cable concentrations on the inner columns presented a novel feature in the design of bridge towers. The efficiency of this design in distributing the stresses through its members to the piers had to be established beyond all reasonable and possible doubt. In developing the design of the towers it was fully realized that the mathematical analysis and the proportioning of the various members are based on tacit assumptions that pre-suppose the unison of the behavior of the constituent members, and especially the continuous

elastic action of the frame. The basis of the design also assumes that the modulus of elasticity of each of the steel members is practically the same in all parts, which assumption underlies the design of all engineering structures for the same materials.

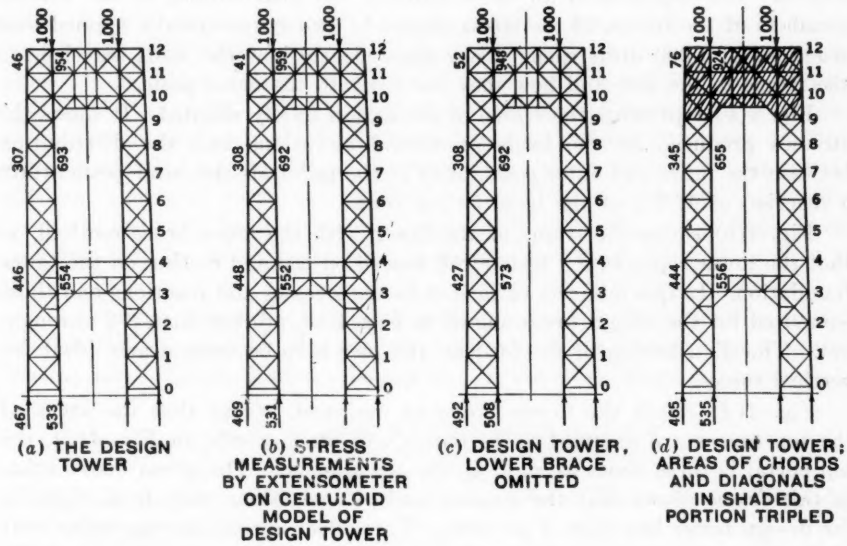


FIG. 4.—VARIATIONS IN DESIGN OF TOWERS.

It was thought desirable, therefore, to test the sensitivity of the tower frame by varying the assumed elastic conditions of its various members and the character and proportions of the braces that connect the two bents. In

this way, a measure was obtained of the sensitiveness of the stresses in the tower to such variations. These studies furnished interesting information as to the behavior of statically indeterminate structures of this type and showed their inherent stability. A number of these studies are shown on Fig. 4. The numbers in all cases indicate the distribution, in the various members of the frame, of the stress caused by two symmetrically applied load reactions of 1 000 units each. They show clearly how the loads travel down the frame to the pier and how they are fed into the outer posts.

Fig. 4(*b*) represents the frame of the design finally adopted. It shows the stresses produced by the loads at several elevations and the distribution between the inner and outer posts, until at the pedestals the outer posts attain a reaction of 46.7% of the ideal 50 per cent.

Fig. 4(*c*) shows the frame of the tower with the lower brace omitted, so that the tower represents a frame well braced on top and resting on the tower foundation. In this case the variations in the stresses and reactions and those computed for the design tower shown in Fig. 4(*b*), are less than 3% throughout. The distribution of the load on the pier here becomes nearly ideal for vertical forces.

Fig. 4(*d*) shows the tower frame as designed, except that the areas of the top braces are assumed to be tripled; in other words, in Fig. 4(*d*), the top portal is three times as rigid as the one adopted. The stress distribution in this frame shows that the stresses and pier reactions vary from those in the design tower less than 4 per cent. This effect appears in the upper part of the tower and is almost lost in the lower part. It serves to prove the sufficient rigidity of the top braces as built and further proves that increased rigidity would improve the distribution of the stresses only slightly.

Fig. 4(*e*) shows a tower frame with all three braces hinged so that they cannot transmit any bending moments. The variations in the stresses and reactions of this frame have been found to vary from those of the design tower (Fig. 4(*a*)) less than 1 per cent.

Fig. 4(*f*) shows a frame with the simplest and most flexible connection between the two bents. All the braces are replaced by a single strut at the top, hinged at both ends. This is a frame statically indeterminate in the first degree. It represents the lowest extreme variation in the bracing. Even in this case of greatest flexibility the maximum variation of the stresses from those of the design tower does not exceed 19 per cent.

Fig. 4(*g*) represents the tower as designed except that the assumption was made here that the modulus of elasticity of all diagonals between the posts is only one-half its value; or (what is the same), that their respective sectional areas are only one-half those in the design tower. This is also equivalent to a slip of 0.1 in. in the riveted connection at one end of each diagonal. The variations in the stresses and reactions resulting from this assumption are not more than 3% from those in the design tower. The studies of the effect of diagonals on the stress distribution are of importance in the design, because here the connections are affected by rivets only and are not made on abutting ends.

To check the stress distribution roughly, the tower may be considered as consisting of two tower bents for the lower nine panels, with a comparatively rigid truss resting on the top of these bents. The reaction applied on top of this truss would be distributed by it to the posts in the same manner as any truss transmits loads to its piers. An analysis of the stresses from this point of view showed that the truss as actually proportioned could safely transmit at least 40% of the load to the outer posts, whereas the elastic computations on which the design is based, require only about 30% to be transmitted at that point. Pushing this gross assumption further by assuming that all the diagonals in the bents in the lower nine panels have been omitted, without affecting the stability of the tower, the outer posts would thus sustain 40% and the inner posts 60% of the reaction at the top. Actually, the outer posts are designed for a stress varying from 39% at Point 9 to 61% at the pedestal, and the inner posts for a stress from 70% at Point 9 to 61% at the bottom.

To obtain a comprehensive picture of the results of these studies similar to what the biologist would term "the mutation of the species," the resulting stresses were plotted as shown in Fig. 5. In the same diagram is shown the

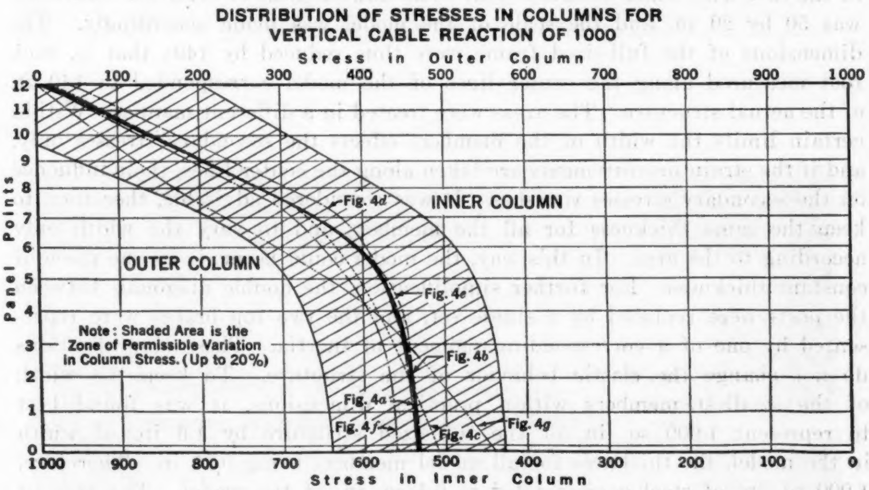


FIG. 5.—CURVES SHOWING RELATIVE DISTRIBUTION OF A VERTICAL CABLE REACTION OF 1000 BETWEEN INNER AND OUTER COLUMNS OF TOWER (SEE FIG. 4).

hatched zone enclosing a variation of stress of 20 per cent. It may be called the zone of permissible stress variations from computed theoretical stress values. Within this zone are found the extremely improbable and practically impossible variations that have been the objectives of this study.

The studies of the efficiency of the design show definitely that although the tower is statically indeterminate to a high degree, it is definite and steady in its behavior and is little disturbed by variations from the assumptions on which the elastic computations are based.

Stress-Strain Measurements on Model of Tower

As indicated by the foregoing studies, the design of the steel towers for this bridge presented a novel feature in the planning of towers for large bridges under special conditions. The analytical treatment of the towers involves the handling of many unknowns and requires elaborate arithmetical computations. It was deemed desirable, therefore, to present a physical object and to obtain sufficient vertification of the action of the frames so that their elastic behavior under load could be demonstrated. As the main object of the test was to observe the distribution of stress between the inner and outer posts, it was sufficient to consider one of the four frames only. Such a frame could not act as a compression member, of course, and would fail in buckling. It was concluded, therefore, that the object of the test would be attained if the frame were to be reversed in position, with the top down, and the reactions applied to it as loads, thus putting the entire frame in tension. Such a model should be of a uniform homogeneous material, preferably of one piece. It should behave elastically the same as a complete structure, or as nearly as it can be made to do so.

Celluloid was selected for the model because of its uniformity and because of the ease with which it can be cut. The largest sheet of celluloid obtainable was 50 by 20 in. and the scale of the model was made accordingly. The dimensions of the full-sized frame were thus reduced by 140; that is, each foot measured along the center lines of the model corresponded to 140 ft. of the actual structure. The areas were treated in a different manner. Within certain limits the width of the members effects the secondary stresses only, and if the strain measurements are taken along the center lines, their influence on the secondary stresses vanishes. It was considered advisable, therefore, to keep the same thickness for all the members and to vary the width only according to the area. In this way, the model could be made in one piece of constant thickness. For further simplification, the double diagonals between the posts were replaced by a single set, and the two top braces were represented by one of a corresponding moment of inertia. These simplifications do not change the elastic behavior of the structure. To keep the width of the smallest members within practical dimensions, it was found best to represent 1 000 sq. in. of the full-sized structure by 1.6 in. of width in the model, the thickness for all model members being 0.25 in. Therefore, 1 000 sq. in. of steel corresponded to 0.4 sq. in. of the model. The area of reduction thus was 1: 2 500.

Incidentally, the celluloid model presented a good picture of the distribution of the material in the actual structure because the width of the members was proportional to their cross-sectional areas. To eliminate, entirely, any possibility of longitudinal flexure from compressive forces, the model was inverted, allowed to swing freely, and the load reactions were applied at the proper places as tensile forces, as shown in Fig. 6. It was subjected to two equal loads of 62.5 lb. which produced a greatest unit stress in the frame of 108 lb. per sq. in. This stress is well within the elastic limit of the celluloid.

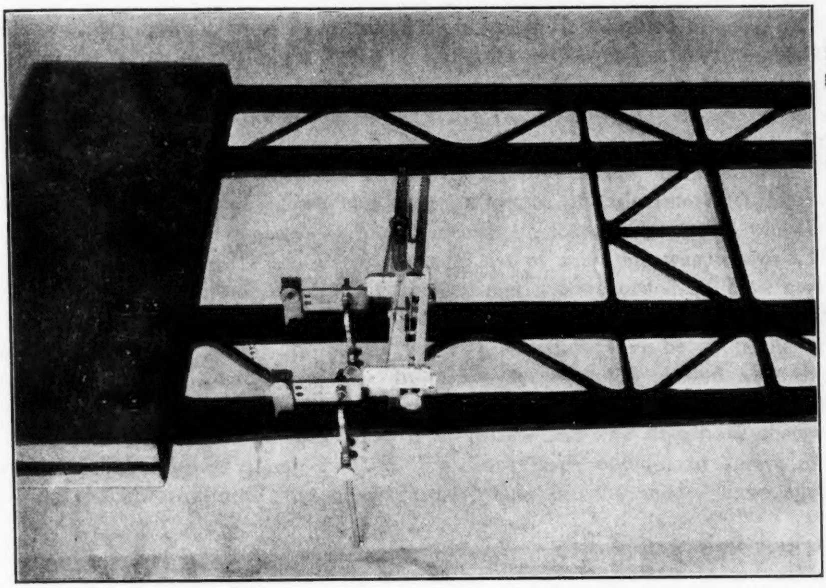


FIG. 7.—TENSOMETERS IN POSITION DURING TEST

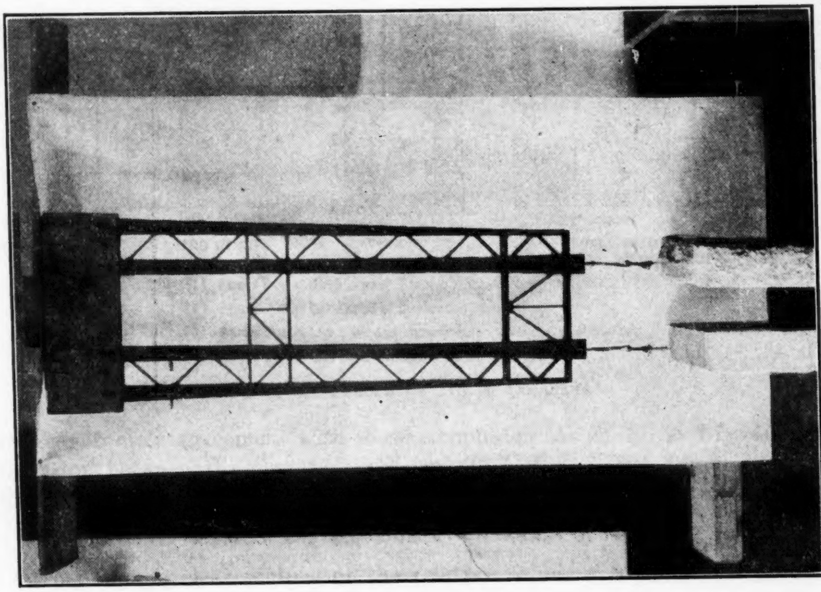


FIG. 6.—CELLULOID MODEL OF TOWER, INVERTED AND SUBJECT TO TENSILE LOADS.

The instruments used to measure the distribution of stresses are based on Hooke's law; namely, the elongation of the material is assumed to be in direct proportion to its unit stresses. Two tensometers with a magnification of 1 000 were used with very satisfactory results (see Fig. 7). The instruments are easy to handle and give sufficiently accurate results.

Two instruments were placed at the same level, one on the center line of the inner posts and the other on the center line of the outer posts of the same side of the model. Both had a gauge length of 1 decimeter (3.94 in.). The first tensometer readings were taken with no load. The model was then subjected to the two loads, and the second set of readings was taken. The load was removed, and a third set of readings was recorded. These corresponded closely to the first readings. The elongation of the first tests is the average of the foregoing differences; that is, the difference between the first and the second readings and the difference between the second and third readings. To obtain a fair average the test was repeated four times.

The elongations were magnified 1 000 times and the distances observed attain an average of about 1 in., a difference large enough to derive close results. The results of the observations made on the model have shown

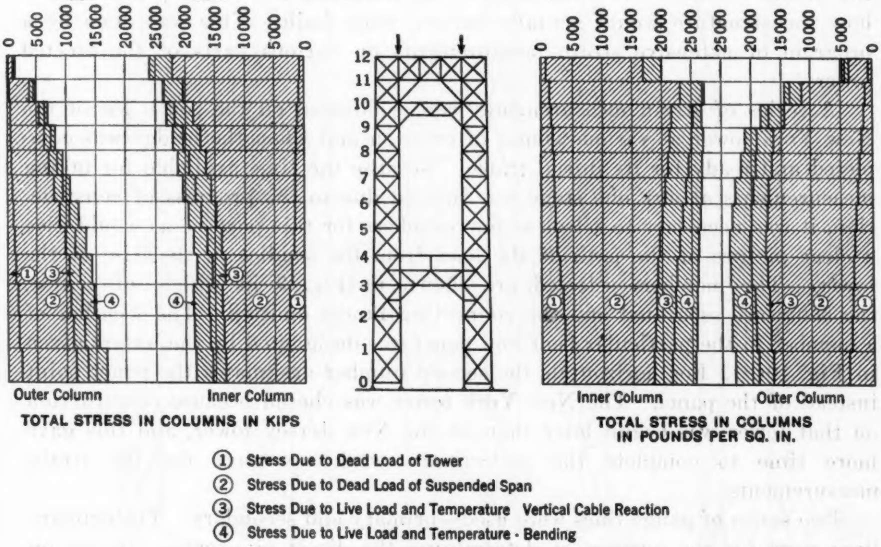


FIG. 8.—STRESS IN TOWER COLUMNS.

stresses in close agreement with those computed. As shown in Fig. 4(*b*), the variation between the observed and computed stresses was found to be less than 1 per cent.

STRESS DISTRIBUTION IN TOWER COLUMNS

To obtain a clear picture of the relative values of the stresses produced in the twelve tower panels by the various loads, the diagrams shown in Fig. 8 were drawn. The left side represents the total stresses in the outer and inner

columns that are produced by the dead load of the towers, the dead load of the suspended span, the live load and the temperature due to the vertical cable reaction, and the live load and the temperature due to bending. The right side represents the unit stresses due to these respective causes. The diagrams in Fig. 8 show at sight the duties that the tower members must perform to sustain the forces that act on the tower. From the diagram of the unit stresses the relative importance can also be seen of the action of these forces.

Strain Measurements

The determination of the stresses by the analytical process, the studies of the elastic behavior and stability of the frames by what may be termed scholastic methods of reasoning, and the observations made on a simple model with the aid of modern apparatus, were all means of predicting the stress-strain behavior of the tower. They are all available tools of science, reason, and experience to foretell the action of a structure designed and erected in accordance with certain premises. They furnished sufficient concording information to heighten confidence and trust in the stability and the safety of the planned towers. It remained for the engineers to verify by observation how the structure would actually behave when built. This was done by a program of extensive strain measurements on various parts of the erected towers.

A series of strain measurements were conducted on the south leg of the New York tower at various stages of erection and when the bridge was completed and ready for its initial traffic. Because the time available for taking measurements at any one stage was limited due to the progress of construction, it was necessary to select as few members for this purpose as would give a clear picture of the path of the load from the saddles to the base of the tower. The members selected are shown in Fig. 9 in which, of course, the bents are separated and the connecting braces omitted. The location of a member in the particular bent and panel was designated by the letters given in Fig. 10(*a*). For horizontals the second number designates the panel point instead of the panel. The New York tower was chosen because construction on that tower was begun later than on the New Jersey tower, and this gave more time to complete the preliminary work necessary for the strain measurements.

Two series of gauge lines were used—primary and secondary. The primary lines were for the purpose of determining the direct or average stresses on the sections, and the secondary lines for determining the secondary bending stresses, or the distribution of stress across the observed sections. The primary series consisted of 20-in. gauge lines in all the members selected. The lines were located as nearly as possible to the middle of each member, care being taken to keep them away from reinforcing plates, splices, or other connections that might introduce local stresses.

Fig. 10(*b*) shows the location of the individual gauge lines in the section of the tower columns. Lines 1 to 8 at the mid-height of the column are primary gauge lines; Lines 1 to 8 at the top and bottom of the column are

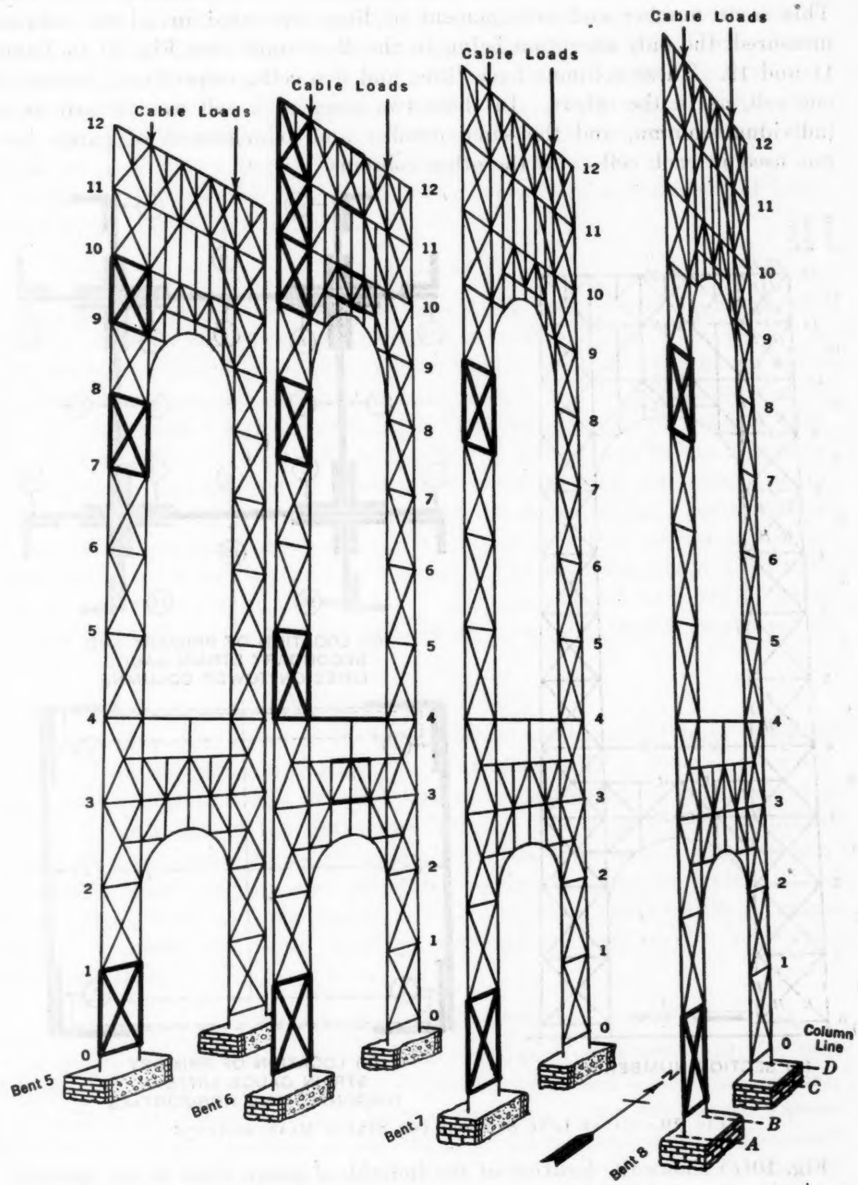


FIG. 9.—MEMBERS OF NEW YORK TOWER SELECTED FOR STRESS IN MEASUREMENTS.

secondary gauge lines; and Lines 9 to 20, inclusive, are secondary gauge lines. This same number and arrangement of lines was used in all the columns measured, the only exception being in the *B*-columns (see Fig. 9) in Panels 11 and 12. These columns have three and five cells, respectively, instead of one cell, as in the others. In these two cases each cell was treated as an individual column, and the same number and arrangement of gauge lines was used in each cell as in the other columns.

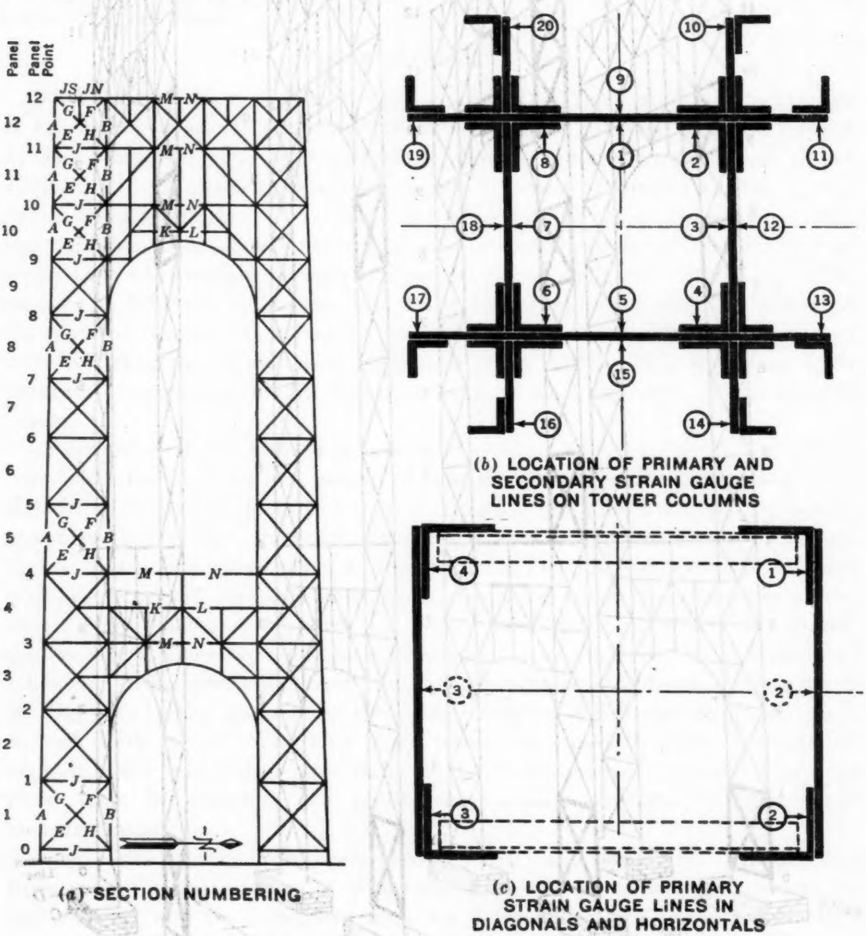


FIG. 10.—GAUGE LINE SYMBOLS FOR STRESS MEASUREMENTS.

Fig. 10(*c*) shows the location of the individual gauge lines in the sections of the diagonals and horizontals. The only variation in this arrangement occurred in the deep horizontals in Panel 12 and in the top portal where an additional gauge line was placed in the middle of the web on each side. The gauge lines shown dotted were additional ones assigned to deep horizontals in Panel 12 and in the top portal.

The secondary gauge lines were placed on the *B*-columns in Panel 1. They were 10-in. lines located at the top and bottom of each column, both inside and outside (see Fig. 10(*b*)). To determine Poisson's ratio an additional set of four 10-in. gauge lines was placed in Column 51*B* at right angles to the primary set at that point to measure the strain at right angles to the axis of the column. In addition, a set of 20-in. gauge lines was placed on the outside of each *B*-column in Panel 1 opposite the 20-in. primary set on the inside. All these lines were located so as to minimize the effect of local stresses from splices or connections.

Gauge Holes

A gauge line consists of two holes spaced the proper distance apart. To keep the distance within proper limits magnetic templets were used. A 6-volt storage battery supplied the current for the templet. The gauge holes were drilled through the templets, by means of a No. 55 combination drill and a 60° countersink of high-speed steel. The drill was run by a ¼-h. p. electric motor and flexible shaft. After a hole was drilled, and before removing the templet, it was finished by inserting a hardened steel pin with a point having the same angle (55°) as the legs of the strain gauge. Striking this pin lightly with a hammer smoothed out any irregularities or fins and made a good seat for the legs of the strain gauge. The holes were kept filled with vaseline and covered with adhesive tape. The gauge holes were drilled while the members were in the railroad yard or as soon after they were erected as possible.

Measuring Equipment

The strain gauges operated by hand were of an improved form designed by R. S. Johnston M. Am. Soc. C. E. Combined with a 10 to 1 lever the dial read directly to 0.0001 in. and was easily estimated to one-tenth of a division. The range of the gauge was 0.04 in. The gauges were calibrated throughout their entire range at various intervals by means of a micrometer microscope, which in turn was calibrated against a standard scale. Three strain gauges were used, one having a 10-in., and two having a 20-in. gauge length. Temperatures were read by mercury as well as by resistance thermometers.

Method of Taking Readings

At sections where readings were taken fixed platforms were built for that purpose. Movable ladder jacks and platforms were used inside the columns. All readings were taken at night because exposure to the sun caused uneven temperatures in the members during the day. In all cases strain-gauge readings were never taken until at least three hours after sunset. The stresses deduced from the strain measurements were based on a modulus of elasticity of 30 000 000. The average modulus of elasticity for twenty-nine test specimens from material representing the columns was 29 800 000. The minimum value was 28 800 000 and the maximum, 31 000 000. The study of the reliability

of the measurements obtained led to the conclusion that as a whole the errors in the measured results were within the following ranges: For diagonals and horizontal, ± 200 lb. per sq. in., with a maximum of 800 lb. per sq. in.; and, for columns, ± 500 lb. per sq. in., with a maximum of 1 300 lb. per sq. in.

TABLE 1.—COMPARISON OF COMPUTED STRESSES WITH THOSE MEASURED BY STRAIN GAUGES

(Units are kips, or thousands of pounds, per square inch)

M or C*	Columns						Diagonals						Horizontals					
	Member †	Stages ‡					Member †	Stages ‡					Member †	Stages ‡				
		1	2	3	4	5		1	2	3	4	5		1	2	3	4	5
M	51A	−5.1	−10.4	−12.6	−18.0	−14.4	81H	−1.4	−1.0	−3.0	−1.2	−1.6	50J	+2.0	.....	+2.7	+2.7	+1.0
C		−4.5	−10.3	−12.4	−17.1	−13.4		−0.4	−0.8	−2.2	−1.8	−3.4		+0.2	.....	+0.3	+1.2	0
M	61A	−3.9	−9.0	−11.4	−15.0	−14.6	65E	−2.1	.....	−10.5	.....	−12.8	60J	+0.9	.....	+1.5	+1.0	0
C		−4.0	−8.6	−11.5	−14.5	−13.4		.....	.....	.....	.....	.....		+0.1	.....	+0.1	+0.4	0
M	71A	−3.4	−7.2	−11.0	−10.2	−15.0	65F	−2.4	.....	−11.1	.....	−13.2	70J	+2.0	.....	+3.0	+2.7	+0.6
C		−3.4	−7.0	−10.7	−11.9	−13.4		−2.5	.....	−10.5	.....	−13.0		−0.1	.....	−0.1	+0.4	0
M	81A	−1.4	−4.5	−9.6	−12.3	−13.6	65G	+0.5	.....	+2.6	.....	+3.8	80J	+3.4	.....	+3.0	+3.0	+2.0
C		−3.0	−5.4	−9.3	− 9.4	−13.5		.....	.....	.....	.....	.....		−0.2	.....	−0.3	+1.2	0
M	51B	−6.2	−11.7	−14.1	−19.8	−14.7	65H	+0.3	.....	+2.6	.....	+2.8	51J	+2.8	.....	+7.5	+9.9	+8.4
C		−5.0	−12.0	−14.4	−19.6	−15.5		+0.3	.....	+2.8	.....	+3.7		+2.6	.....	+7.3	+11.1	+8.0
M	61B	−3.8	−8.7	−12.4	−15.8	−14.8	58E	−1.6	−6.3	−9.2	.....	−12.2	61J	+1.2	.....	+7.2	+9.0	+8.2
C		−4.5	−10.0	−13.4	−16.7	−15.5		.....	.....	.....	.....	.....		+2.3	.....	+6.9	+8.7	+8.0
M	71B	−2.6	−6.4	−11.2	−12.2	−13.8	58F	−1.8	−6.3	−9.2	.....	−12.4	71J	+2.1	.....	+7.0	+7.5	+8.4
C		−4.1	−8.7	−12.3	−13.8	−15.5		−2.1	−6.2	−8.9	.....	−11.2		+2.0	.....	+6.4	+7.2	+8.0
M	81B	−2.2	−5.7	−11.2	−10.5	−15.0	58G	0	+0.6	+1.5	.....	+2.7	81J	+1.6	.....	+6.0	+6.4	+9.0
C		−3.6	−6.4	−11.3	−11.0	−15.6		.....	.....	.....	.....	.....		+2.6	.....	+6.0	+5.7	+8.1
M	65A	−3.2	.....	−11.6	.....	−14.4	58H	+0.3	+1.0	+2.0	.....	+2.7	64J	+1.6	.....	+6.0	.....	+7.5
C		−3.2	.....	−11.4	.....	−13.8		+0.5	+1.9	+3.0	.....	+3.6		+1.3	.....	+5.5	.....	+6.8
M	65B	−3.6	.....	−12.4	.....	−14.8	68E	−1.6	−5.8	−8.8	.....	−11.7	65J	+1.8	.....	+7.2	.....	+8.6
C		−3.2	.....	−12.6	.....	−15.4		.....	.....	.....	.....	.....		+1.5	.....	+5.8	.....	+7.1
M	58A	−1.6	−6.3	−9.2	.....	−11.7	68F	−1.6	−6.2	−9.4	.....	−12.6	57J	+1.6	+5.1	+6.6	.....	+8.1
C		−2.8	−7.6	−10.6	.....	−12.6		−2.0	−6.1	−9.0	.....	−11.3		+1.4	+4.0	+5.4	.....	+6.8
M	68A	−1.5	−6.3	−9.6	.....	−12.3	68G	+1.2	+2.7	+3.3	.....	+4.0	67J	+2.1	+5.0	+6.3	.....	+8.1
C		−2.5	−7.2	−10.4	.....	−12.7		.....	.....	.....	.....	.....		+1.3	+3.8	+5.5	.....	+6.9
M	78A	−2.1	−6.6	−10.5	.....	−13.4	68H	+1.6	+3.0	+3.0	.....	+3.6	77J	+1.8	+4.8	+6.6	.....	+8.4
C		−2.2	−6.8	−10.2	.....	−12.8		+0.6	+1.9	+2.9	.....	+3.6		+1.1	+3.7	+5.6	.....	+7.0
M	88A	−0.3	−5.0	−8.8	.....	−12.8	78E	−1.0	−5.4	−8.2	.....	−11.0	87J	+1.2	+4.2	+6.0	.....	+8.7
C		−1.9	−6.3	−10.0	.....	−12.9		.....	.....	.....	.....	.....		+1.0	+3.5	+5.7	.....	+7.0
M	58B	−2.7	−8.6	−12.6	.....	−15.6	78F	−0.4	−5.6	−8.2	.....	−11.1	58J	−0.2	+2.2	+2.7	.....	+4.0
C		−2.8	−8.8	−12.2	.....	−15.7		−1.9	−6.1	−9.1	.....	−11.3		+1.4	+4.0	+5.5	.....	+7.2
M	68B	−2.8	−8.8	−12.4	.....	−16.5	78G	−0.2	+2.7	+3.3	.....	+3.2	68J	+1.2	+3.3	+5.0	.....	+6.0
C		−2.6	−8.6	−12.5	.....	−15.8		.....	.....	.....	.....	.....		+1.3	+4.0	+5.7	.....	+7.3
M	78B	−3.0	−8.7	−13.0	.....	−17.1	78H	+1.2	+2.7	+3.6	.....	+3.2	78J	+1.8	+4.0	+5.4	.....	+6.9
C		−2.4	−8.3	−12.8	.....	−15.9		+0.7	+2.0	+2.8	.....	+3.6		+1.2	+4.0	+6.0	.....	+7.3
M	88B	−2.2	−8.4	−13.5	.....	−17.2	88E	−1.2	−5.8	−8.8	.....	−11.7	88J	+1.6	+3.0	+4.0	.....	+6.0
C		−2.2	−8.1	−13.1	.....	−16.0		.....	.....	.....	.....	.....		+1.0	+3.9	+6.2	.....	+7.4
M	510A	−2.0	.....	−8.2	.....	−10.8	88F	−1.2	−5.4	−8.7	.....	−11.1	59J	+0.3	.....	+3.9	.....	+5.7
C		−1.9	.....	−8.5	.....	−10.7		−1.8	−6.0	−9.2	.....	−11.3		+0.5	.....	+3.8	.....	+5.3
M	510A	−2.8	.....	−12.0	.....	−15.3	88G	+1.5	+2.1	+2.7	.....	+2.8	69J	+0.9	.....	+5.2	.....	+7.3
C		−2.2	.....	−11.6	.....	−15.8		.....	.....	.....	.....	.....		+0.5	.....	+4.0	.....	+5.3
M	610A	−1.6	.....	−8.8	.....	−11.2	88H	+1.2	+2.1	+2.6	.....	+2.6	510J	+0.2	.....	+5.6	.....	+7.6
C		−1.8	.....	−8.6	.....	−10.8		+0.8	+2.0	+2.7	.....	+3.5		+1.4	.....	+6.7	.....	+9.3
M	610B	−3.6	.....	−12.8	.....	−16.5	510E	−1.6	.....	−6.0	.....	−8.7	610J	−1.2	.....	+4.8	.....	+7.2
C		−2.1	.....	−12.3	.....	−15.9		.....	.....	.....	.....	.....		+1.4	.....	+7.2	.....	+9.1
M	611A	−1.8	.....	−6.4	.....	−10.4	510F	−1.5	.....	−6.6	.....	−8.4	611J	+0.3	.....	+3.9	.....	+5.6
C		−1.3	.....	−6.6	.....	−8.4		−1.1	.....	−6.9	.....	−9.2		+0.4	.....	+3.8	.....	+5.1
M	611B	−1.5	.....	−11.5	.....	−15.4	510G	+0.4	.....	+3.0	.....	+4.0	612JN	−0.2	.....	−2.7	.....	−2.8
C		−1.6	.....	−11.1	.....	−14.6												
M	612A	−0.3	.....	−2.4	.....	−3.3	510H	+0.3	.....	+2.4	.....	+3.9	612JS	−0.3	.....	−2.2	.....	−2.7
C		−0.5	.....	−2.4	.....	−3.0		0	.....	+3.2	.....	+3.7		−0.2	.....	−2.4	.....	−3.0
M	612B	−1.4	.....	−9.1	.....	−12.0	610E	−1.6	.....	−6.9	.....	−9.3						
C		−1.1	.....	−9.0	.....	−12.1												

* M denotes measured stresses; C denotes computed stresses.
† Numerals and letter indicate the position of the member in the tower; thus, see, Figs. 9 and 10 (*a*), 50 J, indicates that the member is the horizontal, J, at Punch Point 0, in Bent 5.
‡ See Fig. 11.

TABLE 1.—(*Continued*)

M or C*	Diagonals						Diagonals						Horizontals in Braces					
	Member †	Stages ‡					Member †	Stages					Member †	Stages ‡				
		1	2	3	4	5		1	2	3	4	5		1	2	3	4	5
M	51E	-2.7	-6.8	-7.2	-10.5	-9.8	610F	-1.8	.....	-6.9	.....	-9.3	63K	.....	.....	+1.6	.....	+1.8
C		.....	.....	.....	.....	.....		-1.0	.....	-7.2	.....	-9.2		.....	.....	.....	.....	.....
M	51F	-2.6	-6.4	-6.3	-10.5	-9.9	610G	+0.2	.....	+3.6	.....	+4.8	63L	.....	.....	+1.6	.....	+2.4
C		-3.0	-6.4	-7.9	-10.0	-8.8		.....	.....	.....	.....	.....		.....	.....	+1.0	.....	+1.3
M	51G	-0.3	-0.9	-2.4	-2.2	-0.3	610H	-0.2	.....	+2.7	.....	+4.2	63M	.....	.....	+0.9	.....	+3.0
C		.....	.....	.....	.....	.....		0	.....	+2.9	.....	+3.6		.....	.....	.....	.....	.....
M	51H	-0.9	-1.5	-2.8	-3.0	-1.2	611E	-1.4	.....	-10.4	.....	-14.8	63N	.....	.....	+1.6	.....	+1.8
C		-0.8	-2.8	-3.2	-4.8	-3.3		.....	.....	.....	.....	.....		.....	.....	+1.4	.....	+1.2
M	61E	-2.4	-5.7	-6.2	-9.8	-9.6	611F	-1.4	.....	-10.6	.....	-14.7	64K	.....	.....	+1.8	.....	+2.2
C		.....	.....	.....	.....	.....		-1.6	.....	-10.3	.....	-13.2		.....	.....	.....	.....	.....
M	61F	-2.0	-5.1	-6.2	-9.0	-9.3	611G	+0.6	.....	+6.2	.....	+8.1	64L	.....	.....	+2.4	.....	+3.0
C		-2.9	-5.7	-7.6	-9.3	-8.9		.....	.....	.....	.....	.....		.....	.....	+1.1	.....	+1.3
M	61G	-0.2	-1.4	-2.8	-2.6	-1.4	611H	+0.6	.....	+5.8	.....	+8.4	610K	0	.....	+2.4	.....	+3.8
C		.....	.....	.....	.....	.....		+0.9	.....	+7.0	.....	+8.6		.....	.....	.....	.....	.....
M	61H	-1.2	-1.2	-2.6	-2.6	-1.4	612E	-1.8	.....	-10.8	.....	-15.0	610L	+0.6	.....	+2.2	.....	+4.2
C		-0.7	-2.1	-2.9	-3.8	-3.4		.....	.....	.....	.....	.....		+0.6	.....	+2.0	.....	+2.4
M	71E	-2.1	-5.4	-6.0	-8.6	-10.4	612F	-0.4	.....	-10.2	.....	-13.6	610M	-0.3	.....	+1.4	.....	+2.4
C		.....	.....	.....	.....	.....		-1.4	.....	-11.0	.....	-14.2		.....	.....	.....	.....	.....
M	71F	-1.6	-5.0	-6.2	-8.1	-10.0	612G	+0.9	.....	+8.0	.....	+10.6	610N	-0.3	.....	+4.0	.....	+2.4
C		-2.7	-5.0	-7.3	-8.2	-8.9		.....	.....	.....	.....	.....		+0.1	.....	-2.4	.....	+5.1
M	71G	-0.8	-1.5	-2.8	-1.8	-2.0	612H	+1.2	.....	+7.0	.....	+9.9	611M	-0.9	.....	-2.4	.....	-2.7
C		.....	.....	.....	.....	.....		+0.8	.....	+7.5	.....	+9.3		.....	.....	.....	.....	.....
M	71H	-1.0	-1.5	-2.2	-2.4	-2.1	.....	.....	.....	.....	.....	.....	611N	-0.6	.....	-2.7	.....	-2.4
C		-0.5	-1.4	-2.6	-2.8	-3.4	.....	.....	.....	.....	.....	.....		-0.3	.....	-2.1	.....	-2.8
M	81E	0	-2.7	-5.1	-6.3	-8.8	.....	.....	.....	.....	.....	.....	612M	-1.4	.....	-4.2	.....	-5.0
C		.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....		.....	.....	.....	.....	.....
M	81F	-0.3	-2.8	-4.6	-6.2	-9.0	.....	.....	.....	.....	.....	.....	612N	-1.2	.....	-3.6	.....	-4.5
C		-2.5	-4.3	-7.0	-7.2	-8.9	.....	.....	.....	.....	.....	.....		-0.8	.....	-4.5	.....	-5.6
M	81G	-0.3	-0.5	-2.0	-0.9	-1.4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
C		.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

* M denotes measured stresses; C denotes computed stresses.
† Numerals and letter indicate the position of the member in the tower: thus, see, Figs. 9 and 10 (*a*), 50 J, indicates that the member is the horizontal, J, at Panel Point 0, in Bent 5.
‡ See Fig. 11.

Stages of Loading

Strain measurements were made for five successive stages during the construction of the bridge, which meant five different conditions of loading: (1) With the completed tower and catwalk, or temporary platform, for stringing the cables; (2) Stage 1 with the four cables completed; (3) Stage 2, including the floor steel of the suspended structure; (4), Stage 3, including the completed floor-slab in the main span and three-fourths of that of the side span; and (5) the completed bridge with the single deck in addition to the catwalk. The tower saddles were then in their final position for this stage. These stages are shown diagrammatically in Fig. 11.

The measured and computed stresses for all the sections, for the five stages, are given in Table 1. The measured stress reported is the average for all the primary gauge lines in that section.

Columns

The agreement between measured and computed stress in the columns is very close as shown by Figs. 12 and 13. The distribution of the cable reaction between the inner and outer columns of Bent 6 is shown graphically in Fig. 12. The distribution of the load between the eight columns of the tower leg at Panels 1 and 8 is shown in Fig. 13. The distribution of the total

load between the inner and outer columns is given in Table 2 for measured and computed stresses. Strain measurements were taken on one of the columns for the purpose of determining Poisson's ratio. These measurements con-

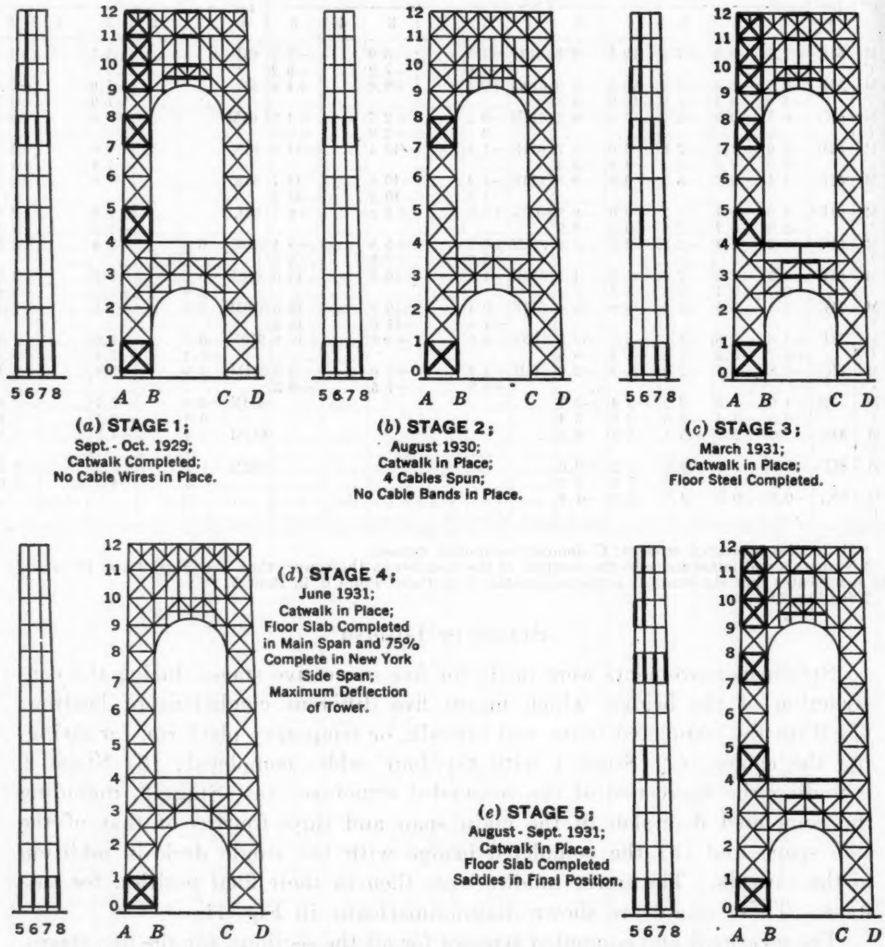


FIG. 11.—DIAGRAMMATIC DESCRIPTION OF STAGES FOR STRAIN-GAUGE MEASUREMENTS.

sisted of one gauge line on each of the four inside plates in Column 51A. The results of these measurements are given in Table 3.

DIAGONALS

The difference between the average measured stress and average computed stress for all the diagonals was 370 lb. per sq. in., with a maximum difference of 2 300 lb. per sq. in. The large differences in measured and computed stress usually occurred when the stresses in the diagonals were small.

TABLE 2.—DISTRIBUTION OF LOAD BETWEEN INNER AND OUTER ROWS OF COLUMNS

Stage	Total Load on Four Outer (A) Columns, in Kips		Total Load on Four Inner (B) Columns, in Kips		Total Load on Outer and Inner Columns, in Kips	
	Measured	Calculated	Measured	Calculated	Measured	Calculated
			Panel 1			
1	9 884	10 694	10 600	12 316	20 484	23 010
2	22 321	22 463	23 270	26 562	45 591	49 025
3	32 010	31 867	35 016	36 802	67 026	68 669
4	39 833	37 967	41 743	43 748	81 576	81 715
5	41 340	38 540	41 741	44 464	83 081	83 004
			Panel 8			
1	2 426	4 146	8 410	7 859	10 836	12 005
2	10 677	12 302	27 120	26 568	37 797	38 870
3	16 806	18 174	40 479	39 772	57 285	57 946
4	......	......	......	......	......	......
5	22 144	22 496	52 192	49 832	74 336	72 328

Horizontals

The horizontal member at Panel Point 12 had two sets of gauge lines. Both the measured and computed stresses for this member checked closely. The difference between the average measured stress and the average computed stress for all horizontals was 300 lb. per sq. in., with a maximum of 3 600 lb. per sq. in.

TABLE 3.—MEASUREMENTS TO DETERMINE POISSON'S RATIO

Item	Primary stress in column	Stress normal to primary stress	Poisson's ratio
1....................	− 6 200	+2 100	0.339
2....................	− 9 200	+3 000	0.326
3....................	−14 100	+4 200	0.298
4....................	−14 700	+4 500	0.303
Average..............	....	......	0.316

Results of Stress-Strain Observations

It has been found that in large members built of plates and angles, such as the columns of this bridge, the intensity of stress is not the same for all parts in any section and may vary along the length of any one part without the application of an intermediate external load. This variation may be as much as 4 500 lb. per sq. in.

The average of the stress measured in part of the shapes making up the section does not necessarily indicate the true average stress in the member. However, if one or more gauge lines were placed on every part making up the section a very accurate measurement of the stress at the given section could be made. What is most important, it has been found that the distribution of load between the inner and outer columns at the base is practically equal. It varies from the computed value about 3½% for the completed structure. The agreement between the measured and computed stresses taken as

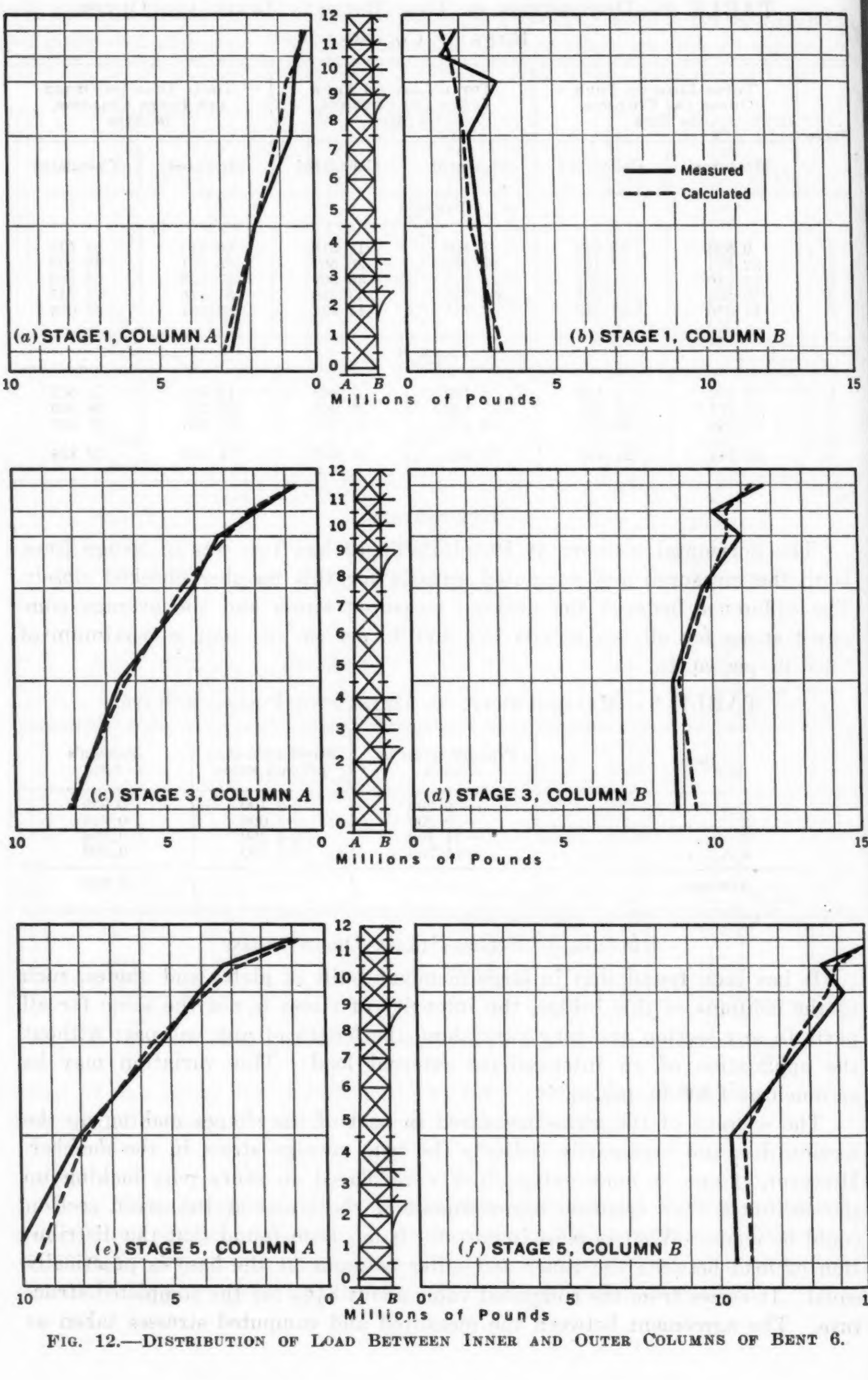


FIG. 12.—DISTRIBUTION OF LOAD BETWEEN INNER AND OUTER COLUMNS OF BENT 6.

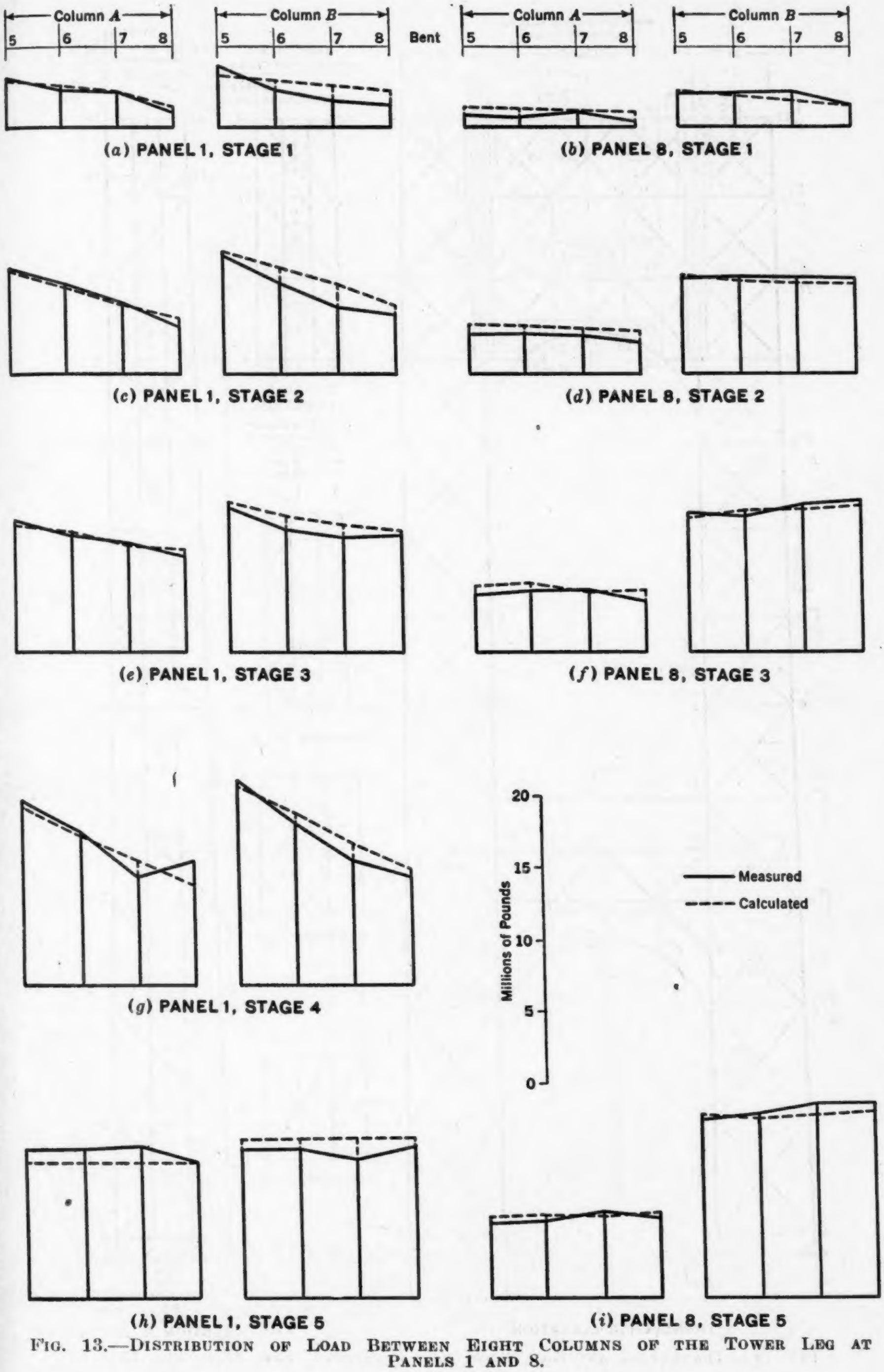


FIG. 13.—DISTRIBUTION OF LOAD BETWEEN EIGHT COLUMNS OF THE TOWER LEG AT PANELS 1 AND 8.

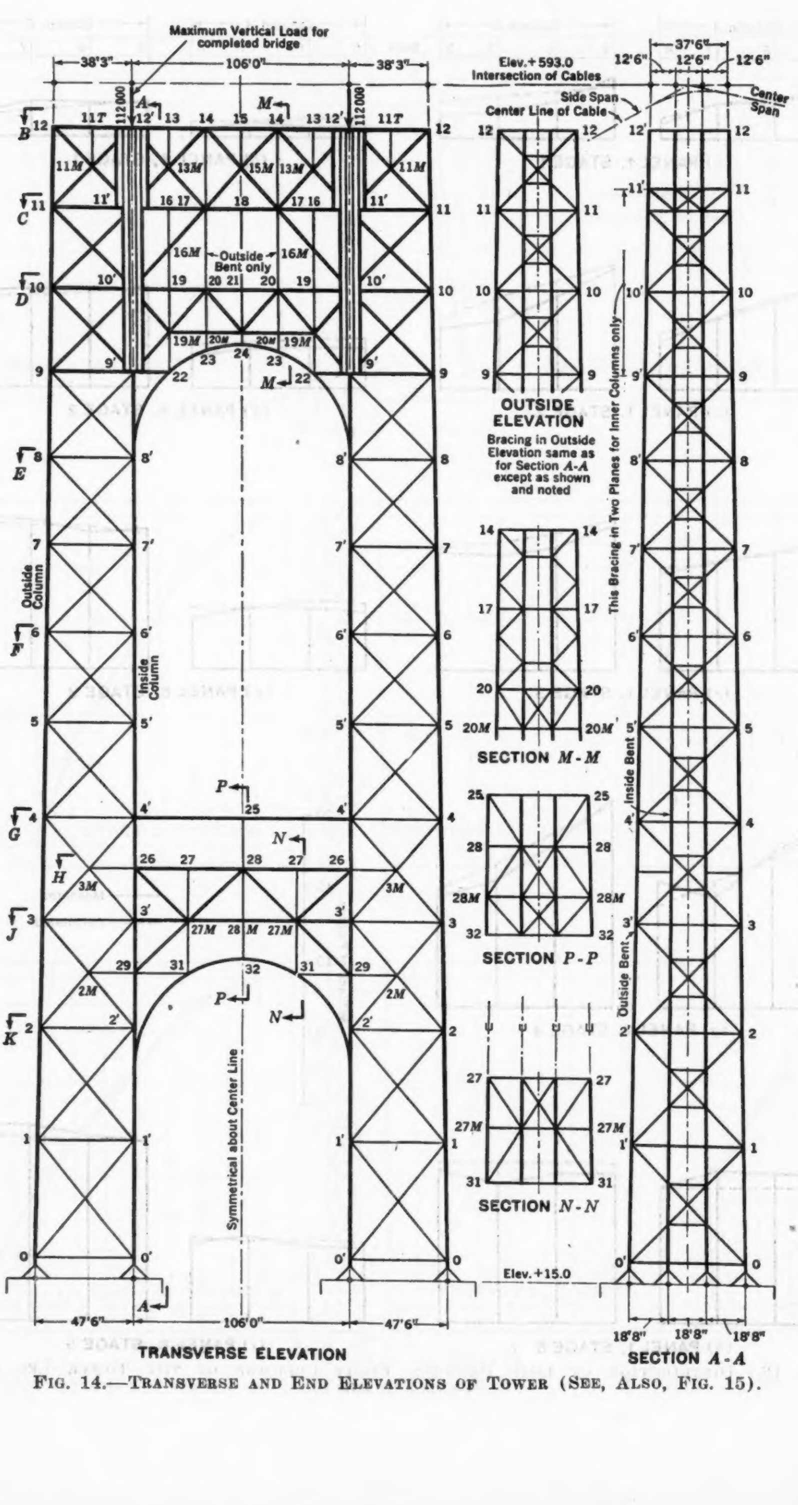


FIG. 14.—TRANSVERSE AND END ELEVATIONS OF TOWER (SEE, ALSO, FIG. 15).

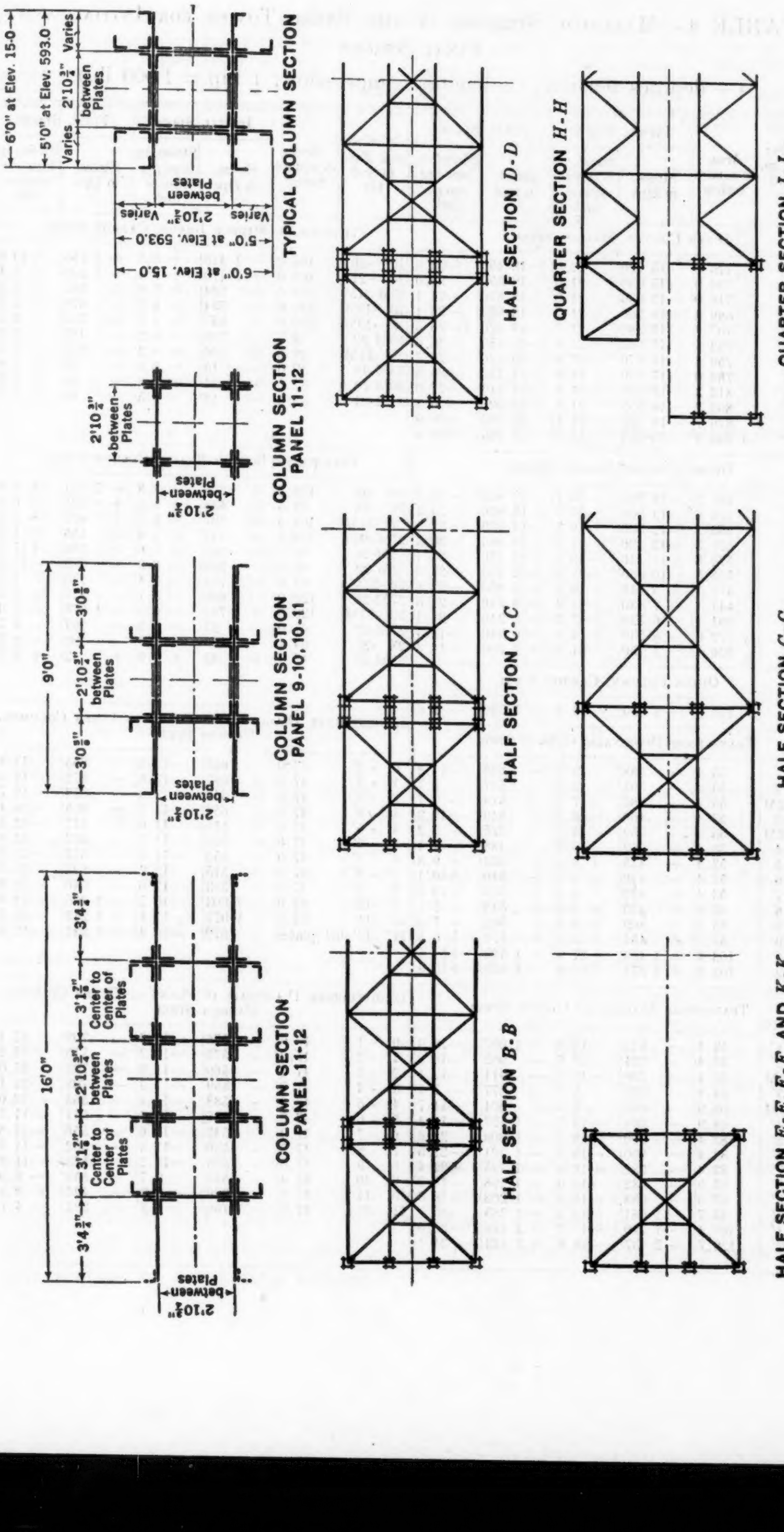


FIG. 15.—DETAILS AND SECTIONS OF TOWER (SEE FIG. 14).

TABLE 4.—MAXIMUM STRESSES IN THE STEEL TOWER FOR INITIAL AND FINAL STAGES

(+ denotes tension; − denotes compression; 1 kip = 1 000 lb.)

Member (see Figs. 14 and 15)	Area, in square inches	Initial Stage: Stress, in kips	Initial Stage: Stress, in kips per square inch	Final Stage: Stress, in kips	Final Stage: Stress, in kips per square inch
INNER COLUMN, SILICON STEEL					
0′- 1′	716.0	−15 720	−22.0	−18 450	−25.8
1′- 2′	716.0	−15 680	−21.9	−18 590	−26.0
2′- 3′	716.0	−15 630	−21.8	−18 670	−26.1
3′- 4′	689.8	−14 990	−21.7	−18 020	−26.1
4′- 5′	707.8	−15 380	−21.7	−18 620	−26.3
5′- 6′	733.5	−15 960	−21.8	−19 480	−26.6
6′- 7′	759.8	−16 570	−21.8	−20 320	−26.7
7′- 8′	786.0	−17 150	−21.8	−21 120	−26.9
8′- 9′	812.3	−17 690	−21.8	−21 910	−27.0
9′-10′	860.2	−18 550	−21.6	−23 090	−26.8
10′-11′	876.7	−18 760	−21.4	−23 560	−26.9
11′-12′	1 296.3	−20 380	−15.7	−25 980	−20.0
OUTER COLUMN; SILICON STEEL					
0- 1	717.7	−14 390	−20.1	−16 860	−23.5
1- 2	674.0	−13 500	−20.0	−15 820	−23.5
2- 3	652.2	−12 590	−19.3	−14 920	−22.9
3- 4	621.2	−12 120	−19.5	−14 460	−23.3
4- 5	574.8	−11 140	−19.4	−13 450	−23.4
5- 6	528.5	−10 060	−19.0	−12 150	−23.0
6- 7	476.1	− 8 940	−18.8	−10 850	−22.8
7- 8	441.1	− 7 830	−17.8	− 9 540	−21.6
8- 9	391.1	− 6 720	−17.2	− 8 240	−21.1
9-10	372.6	− 5 530	−14.8	− 6 840	−18.3
10-11	326.1	− 3 760	−11.5	− 4 660	−14.3
OUTER COLUMN; CARBON STEEL					
11-12	276.8	− 1 050	− 3.8	− 1 280	− 4.6
TRANSVERSE DIAGONALS; CARBON STEEL					
0′ - 1	53.4	− 488	− 9.1	− 628	−11.8
1′ - 2	53.4	− 391	− 7.3	− 517	− 9.7
2′ - 2M	53.4	− 386	− 7.2	− 518	− 9.7
2M- 3	53.4	− 466	− 8.7	− 549	−10.3
3′ - 3M	53.4	− 259	− 4.9	− 275	− 5.2
3M- 4	53.4	− 363	− 6.8	− 488	− 9.1
4′ - 5	53.4	+ 425	+ 8.0	+ 523	+ 9.8
5′ - 6	53.4	+ 438	+ 8.2	+ 540	+10.1
6′ - 7	53.4	+ 439	+ 8.2	+ 543	+10.2
7′ - 8	65.9	+ 430	+ 6.5	+ 532	+ 8.1
8′ - 9	65.9	+ 402	+ 6.1	+ 502	+ 7.6
9′ -10	87.7	+ 454	+ 5.2	+ 574	+ 6.5
10′ -11	103.4	+ 1 189	+11.5	+ 1 505	+14.5
11′ -12	103.4	+ 1 271	+12.4	+ 1 633	+15.8
TRANSVERSE DIAGONALS; CARBON STEEL					
0 - 1′	53.4	− 815	−15.3	− 1 007	−18.9
1 - 2′	53.4	− 724	−13.6	− 902	−16.9
2 - 2M	53.4	− 730	−13.7	− 914	−17.1
2M- 3′	53.4	− 302	− 5.7	− 377	− 7.1
3 - 3M	68.9	− 655	− 9.5	− 804	−11.7
3M- 4′	68.9	− 904	−13.1	− 1 127	−16.4
4 - 5′	83.4	− 1 000	−18.7	− 1 209	−22.6
5 - 6′	83.4	− 990	−18.5	− 1 201	−22.5
6 - 7′	83.4	− 982	−18.4	− 1 195	−22.4
7 - 8′	65.9	− 982	−14.9	− 1 198	−18.2
8 - 9′	65.9	− 988	−15.0	− 1 208	−18.3
9 -10′	87.7	− 1 081	−12.3	− 1 363	−15.5
10 -11′	103.4	− 1 790	−17.3	− 2 274	−22.0
11 -12′	129.7	− 2 437	−18.8	− 3 123	−24.1
TRANSVERSE MIDDLE BRACE; CARBON STEEL					
10′ -20	166.0	+ 1 415	+ 8.5	+ 1 756	+10.6
20 -21	166.0	+ 1 170	+ 7.1	+ 1 506	+ 9.1
19M-21M	202.0	+ 684	+ 3.4	+ 845	+ 4.2
10′ -19M	130.0	+ 554	+ 4.3	+ 676	+ 5.2
9′ -19M	130.0	+ 557	+ 4.3	+ 764	+ 5.9
19M-20	78.7	− 329	− 4.2	− 415	− 5.3
20 -21M	39.0	+ 240	+ 6.2	+ 326	+ 8.4
19M-19	39.0	− 15	− 0.4	− 15	− 0.4
20M-20	39.0	+ 34	+ 0.8	+ 34	+ 0.8
21M-21	39.0	− 18	− 0.5	− 18	− 0.5
TRANSVERSE BOTTOM BRACE; CARBON STEEL					
4′ -25	179.0	+ 675	+ 3.8	+ 1 211	+ 6.8
26 -28	95.5	+ 438	+ 4.6	+ 605	+ 6.3
3′ -27M	108.5	+ 397	+ 3.7	+ 476	+ 4.4
27M-28M	108.5	+ 147	+ 1.4	+ 155	+ 1.4
3M-26	83.4	+ 407	+ 7.6	+ 598	+11.2
2M-29	69.0	− 568	− 8.2	− 783	−11.3
26 -27M	91.0	− 124	− 1.4	− 130	− 1.4
27M-28	120.0	− 608	− 5.1	− 829	− 6.9
29 -27M	120.0	− 781	− 6.5	− 1 078	− 9.0
27M-27	47.0	− 21	− 0.5	− 197	− 4.2
28 -25	64.4	− 406	− 6.3	− 406	− 6.3
28M-28	47.0	+ 42	+ 0.9	+ 42	+ 0.9
LONGITUDINAL DIAGONALS IN PLANE OF INNER COLUMNS; CARBON STEEL					
0′ - 1′	47.0	− 646	−13.8	− 865	−18.4
1′ - 2′	47.0	− 647	−13.8	− 872	−18.6
2′ - 3′	47.0	− 648	−13.8	− 879	−18.7
3′ - 4′	47.0	− 654	−13.9	− 895	−19.1
4′ - 5′	47.0	− 515	−11.0	− 577	−12.3
5′ - 6′	47.0	− 533	−11.3	− 592	−12.6
6′ - 7′	47.0	− 556	−11.8	− 613	−13.0
7′ - 8′	47.0	− 575	−12.2	− 623	−13.3
8′ - 9′	47.0	− 593	−12.6	− 646	−13.8
9′ -10′	94.0	− 1 003	−10.7	− 1 206	−12.8
10′ -11′	94.0	− 1 072	−11.4	− 1 243	−13.3
11′ -12′	6-1″ plates	− 979	−10.4	− 1 081	−11.5
LONGITUDINAL DIAGONALS IN PLANE OF OUTER COLUMNS; CARBON STEEL					
0 - 1	47.0	− 575	−12.2	− 709	−15.1
1 - 2	47.0	− 575	−12.2	− 715	−15.2
2 - 3	47.0	− 560	−11.9	− 703	−15.0
3 - 4	47.0	− 559	−11.9	− 708	−15.1
4 - 5	47.0	− 538	−11.4	− 565	−12.0
5 - 6	47.0	− 537	−11.4	− 551	−11.7
6 - 7	47.0	− 543	−11.6	− 556	−11.8
7 - 8	47.0	− 530	−11.3	− 532	−11.3
8 - 9	47.0	− 526	−11.2	− 525	−11.2
9 -10	55.4	− 539	− 9.7	− 549	− 8.9
10 -11	47.0	− 390	− 8.3	− 426	− 9.1
11 -12	47.0	− 196	− 4.2	− 222	− 4.7

TABLE 4.—(*Continued*)

Member (see Figs. 14 and 15)	Area, in square inches	Initial Stage: Stress, in kips	Initial Stage: Stress, in kips per square inch	Final Stage: Stress, in kips	Final Stage: Stress, in kips per square inch
Transverse Horizontals; Carbon Steel					
0 - 0′	71.0	0	0	0	0
1 - 1′	47.0	+ 539	+11.5	+ 631	+13.4
2 - 2′	47.0	+ 479	+10.2	+ 559	+11.9
3 - 3′	68.0	+ 725	+10.7	+ 870	+12.8
4 - 4′	82.4	+ 738	+ 9.0	+ 948	+11.5
5 - 5′	47.0	+ 471	+10.0	+ 566	+12.1
6 - 6′	47.0	+ 455	+ 9.7	+ 551	+11.7
7 - 7′	47.0	+ 457	+ 9.7	+ 558	+11.9
8 - 8′	47.0	+ 480	+10.2	+ 587	+12.5
9 - 9′	77.0	+ 617	+ 8.0	+ 761	+ 9.9
10 -10′	77.0	+ 971	+12.6	+ 1 211	+15.7
11 -11′	127.0	+ 878	+ 6.9	+ 1 110	+ 8.8
12 -12′	229.8	+ 1 017	+ 4.0	+ 1 279	+ 5.1
Transverse, Top Brace; Carbon Steel					
12′ -14	304.0	— 2 380	— 7.8	— 2 920	— 9.6
14 -15	304.0	— 2 322	— 7.6	— 2 872	— 8.6
11′ -17	127.0	+ 587	+ 4.6	+ 693	+ 5.5
17 -18	127.0	— 533	— 4.2	— 630	— 5.0
12′ -17	78.0	— 735	— 9.4	— 923	—11.8
11′ -14	78.0	— 231	— 3.0	— 299	— 3.8
14 -15M	78.0	— 315	— 4.0	— 451	— 5.8
17 -15M	78.0	+ 315	+ 4.0	+ 451	+ 5.8
17 -14	47.0	— 172	— 3.7	— 222	— 4.7
10′ -17	96.7	— 1 097	—11.3	— 1 392	—14.4

Member (see Figs. 14 and 15)	Area, in square inches	Initial Stage: Stress, in kips	Initial Stage: Stress, in kips per square inch	Final Stage: Stress, in kips	Final Stage: Stress, in kips per square inch
Longitudinal Horizontals in Plane of Inner Columns; Carbon Steel					
0′ - 0′	71.0	0	0	0	0
1′ - 1′	47.0	+ 368	+ 7.8	+ 457	+ 9.7
2′ - 2′	47.0	+ 370	+ 7.9	+ 462	+ 9.8
3′ - 3′	47.0	+ 379	+ 6.4	+ 475	+ 8.1
4′ - 4′	59.0	+ 378	+ 6.4	+ 477	+ 8.1
5′ - 5′	47.0	+ 375	+ 8.0	+ 475	+10.1
6′ - 6′	47.0	+ 380	+ 8.1	+ 484	+10.3
7′ - 7′	47.0	+ 384	+ 8.2	+ 489	+10.4
8′ - 8′	47.0	+ 388	+ 8.3	+ 495	+10.5
9′ - 9′	118.0	+ 586	+ 5.0	+ 750	+ 6.4
10′ -10′	118.0	+ 781	+ 6.6	+ 1 003	+ 8.5
11′ -11′	154.0	+ 686	+ 4.5	+ 885	+ 5.8
12′ -12′	6-1″ plates.	+ 295	+....	+ 381	+....
Longitudinal Horizontals in Plane of Outer Columns; Carbon Steel					
0 - 0	71.0	0	0	0	0
1 - 1	47.0	+ 320	+ 6.8	+ 400	+ 8.5
2 - 2	47.0	+ 324	+ 6.9	+ 406	+ 8.6
3 - 3	52.0	+ 326	+ 6.2	+ 409	+ 7.7
4 - 4	59.0	+ 331	+ 5.6	+ 417	+ 7.1
5 - 5	47.0	+ 330	+ 7.0	+ 416	+ 8.8
6 - 6	47.0	+ 324	+ 6.9	+ 410	+ 8.7
7 - 7	47.0	+ 315	+ 6.7	+ 400	+ 8.5
8 - 8	47.0	+ 306	+ 6.5	+ 387	+ 8.2
9 - 9	59.0	+ 306	+ 5.2	+ 389	+ 6.6
10 -10	59.0	+ 257	+ 4.4	+ 327	+ 5.5
11 -11	77.0	+ 139	+ 1.8	+ 178	+ 2.3
12 -12	95.0	+ 38	+ 0.4	+ 48	+ 0.5

a whole is very good. The arithmetical average of all the differences was 740 lb. per sq. in. and the algebraic average was 60 lb. per sq. in.

The practically close agreement between the stresses as computed by analysis and those found from extensive and carefully conducted strain measurements on the actual structure as built under five progressive stages of loading is a full justification of the procedure followed in the development of this design. It is one more proof of the correctness of engineering science and reasoning, of the uniformity of the steel used, and of the excellency of its fabrication into members and its erection into the structure.

Indeterminacy.—A few words on indeterminate structures seem to be proper at this point. The term, "statically indeterminate," plainly means that the equations of equilibrium, which are not dependent on the elastic behavior of the structure, are not sufficient to determine the stresses in them. It is easier to analyze a structure that conforms to the purely mathematical conceptions of concentric action, homogeneous material, frictionless hinges, discontinuous members, etc., but such structures do not exist in Nature. Physically, one can only approach them and make allowances for the deficiencies of one's assumptions. Structures that are internally statically indeterminate will be found to be more stable. Correctly analyzed they present a true picture of the acting stresses.

The writer would suggest replacing the term, "statically indeterminate," by the term, "super-static," as more fitting for structures with more members than are merely necessary for equilibrium, so that when a structure requires sixteen elastic equations to be solved to compute its stresses, it may not be considered on first judgment to be sixteen times more unsteady and dangerous. The more that is known of structures the more it is realized that they demand careful and searching study to be designed effectively, and that the ease of computation is no measure of their efficiency.

Stresses and Material

The combination of the loads to be carried by the tower, resulting in greatest stresses is given in Table 4 for both the initial and the final stages. The location of individual members is shown in the framing plan of the tower, Fig. 14, and in Fig. 15. To sustain these large forces two grades of structural steel, which have been used successfully on bridges, were utilized. All main members of the columns were made of silicon steel, except the top members of the outer columns. The bracing (with the exception of a number of diagonals in transverse elevation), all secondary parts, and parts in which stiffness rather than strength was desirable, were made of carbon steel.

In the proportioning of the various members of the towers two distinct loading conditions were considered: One, the initial stage, with the upper deck completed, and the other, the final stage, with the bridge in its final condition and the lower deck added. The initial stage carried 31 400 lb. per ft. of bridge dead load and 4 200 lb. of live load, and the final stage, 39 000 lb. of dead load and 8 000 lb. of live load, resulting in highest cable reactions of 86 000 tons and 112 000 tons, respectively.

The sections of the towers were designed not to exceed the following unit stresses, in pounds per square inch (l = unsupported inches of column or flange; r = radius of gyration; and b = width of flange, in inches):

Silicon Steel:

Tension		20 000
Compression	$27\,000 - 80\frac{l}{r}$, max.	23 000
Bending	$27\,000 - 270\frac{l}{b}$, max.	23 000
Shear		17 000
Bearing		40 000

Carbon Steel:

Tension		20 000
Compression	$20\,000 - 60\frac{l}{r}$, max.	17 000
Bending	$20\,000 - 20\frac{l}{b}$, max.	17 000
Shear		12 500
Bearing		30 000

Power-driven rivets: Shear	12 500
Power-driven rivets: Bearing	25 000

For the final stage—that is, for a dead load of 39 000 lb. and a live load of 8 000 lb. per ft. of bridge—an increase of unit stress was allowed for columns of silicon steel to 28 000 lb., and for columns of carbon steel to 20 500 lb.

The stresses were not to be exceeded for the following two ruling conditions: (1) Dead load plus live load plus temperature, or dead load plus wind;

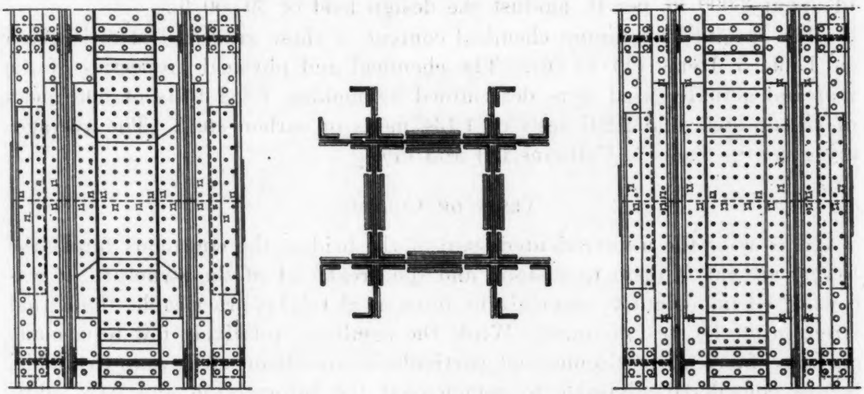


FIG. 16.—TYPICAL COLUMN SPLICE.

and (2) the ratio of the stresses based on the initial stage to those based on the final stage is approximately the same as the ratio between the two sets of unit stresses.

TABLE 5.—PHYSICAL AND CHEMICAL CHARACTERISTICS OF TOWER COLUMN STEEL
(1 kip = 1 000 lb.)

Item	Description	CARBON STEEL			SILICON STEEL	
		SPECIFICATION		Test specimen	Specification	Test specimen
		Structural	Rivet			
(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Chemical Properties:					
(1)	Carbon	....	.....	0.21	0.40	0.35
(2)	Phosphorus, acid	0.06	0.04	0.018	0.06	0.022
(3)	Phosphorus, basic	0.04	0.04	.....	0.04	.....
(4)	Sulfur	0.05	0.045	0.037	0.05	0.037
(5)	Silicon	....	0.45	.....	*	0.27
(6)	Manganese	....	.....	0.50	....	0.78
	Physical Properties:					
(7)	Tensile strength, in kips per square inch	58 to 68	52 to 60	63.6	80 to 95	88
(8)	Yield point (minimum) in kips per square inch	35	30	38.2	45	50.8
(9)	Elongation in 8 in. (minimum percentage)	†	†	28	†	22
(10)	Reduction of area (minimum percentage)	42	52	52	30	43
	Bend Test:‡					
(11)	Material, ¾ in. or less; bend 180°:	Around $D = T$	Flat	.....	Around $D = T$	.....
(12)	Material more than ¾ in. and less than 1¼ in.; bend 180°	Around $D = 1.5\ T$	.....	.....	Around $D = 1.5\ T$	.....

* Not less than 20 per cent.

† Minimum elongation expressed by the ratio, $\dfrac{1\ 500}{\text{tensile strength, in kips}}$

‡ D = inside diameter of the bend; T = thickness of the material.

The column splices were developed for not less than 50% of the stresses for completely milled compression members in bearing. A typical column splice is shown in Fig. 16. Fig. 17 shows a column riveted in the shop and Fig. 18, the top section of the inner column.

It is gratifying to note here that the equivalent dead load for the final stage will be about 36 000 lb. per ft., leaving a margin for contingent loads of about 3 000 lb. per ft. against the design load of 39 000 lb.

The specified maximum chemical content of these grades of steel is given in Table 5, Items (1) to (6). The chemical and physical properties of the material actually used were determined by making 1 884 tests of 895 melts of silicon steel and 2 207 tests of 1 134 melts of carbon steel. The averages are given in Table 5, Columns (5) and (7).

Tests of Columns

In view of the unprecedented span of the bridge, the enormous forces the towers are called upon to sustain, and the great cost of the structure, it was considered important to ascertain by tests, as closely as practicable, the actual strength of the steel columns. While the results of relatively few tests made on large fabricated columns and particularly on silicon steel were available, it was considered advisable to complement the information and gain additional assurance and confidence in a structure of the magnitude of the George Washington Bridge. It was intended to determine the probable strength of the tower columns by testing other columns similar in design and dimensional proportions. Therefore, a program for such large-scale tests was prepared and carried out jointly by the Port of New York Authority and the National Bureau of Standards, in Washington, D. C.[3] R. S. Johnston, M. Am. Soc. C. E., Engineer of Research and Tests, The Port of New York Authority, was in charge of the tests. To this program models of the lower chord section of the Kill van Kull Bridge at Bayonne, N. J., were added later.

The capacity of the 5 000-ton testing machine at the Bureau of Standards, and the fact that the test columns should be similar in design and dimensionally proportioned to the tower-column section of the George Washington Bridge, determined the make-up of the test columns.

The column section selected for test purposes was approximately a half-size reduction of the typical tower column selected for study. The ratio of the respective areas of the test specimens and of the tower column was 1:4.5. The $\frac{l}{r}$ ratios were approximately equal, so that, in view of the relatively low value of $\frac{l}{r}$ of these columns, the fiber stresses due to bending, under loads proportional to their areas, were practically the same. The probable strength of the typical tower column section was thus directly proportional to that of the test column section on the basis of their respective sectional areas.

[3] A complete report of the column tests described herein is being published by the National Bureau of Standards.

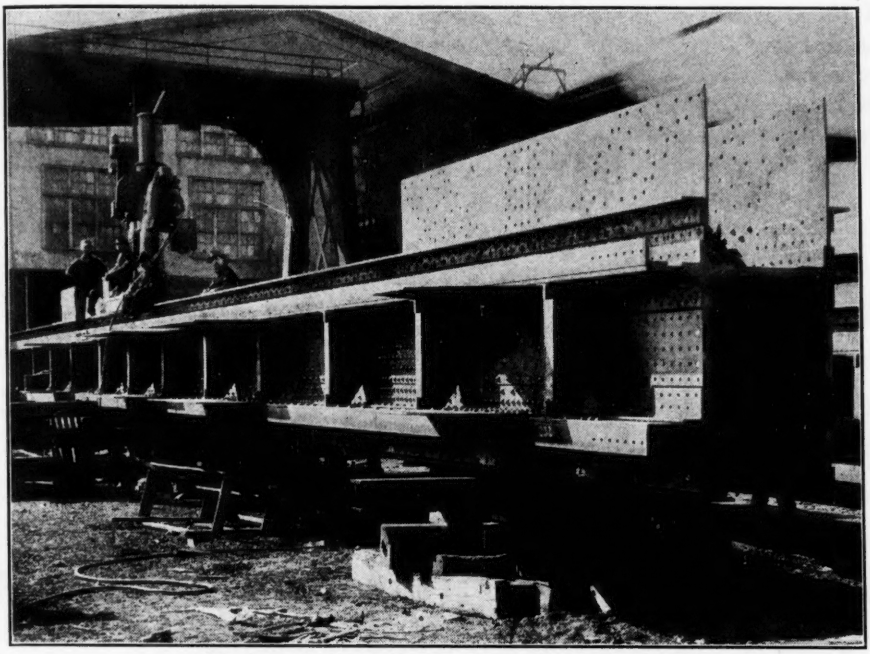


FIG. 17.—VIEW OF COLUMN RIVETED IN THE SHOP.

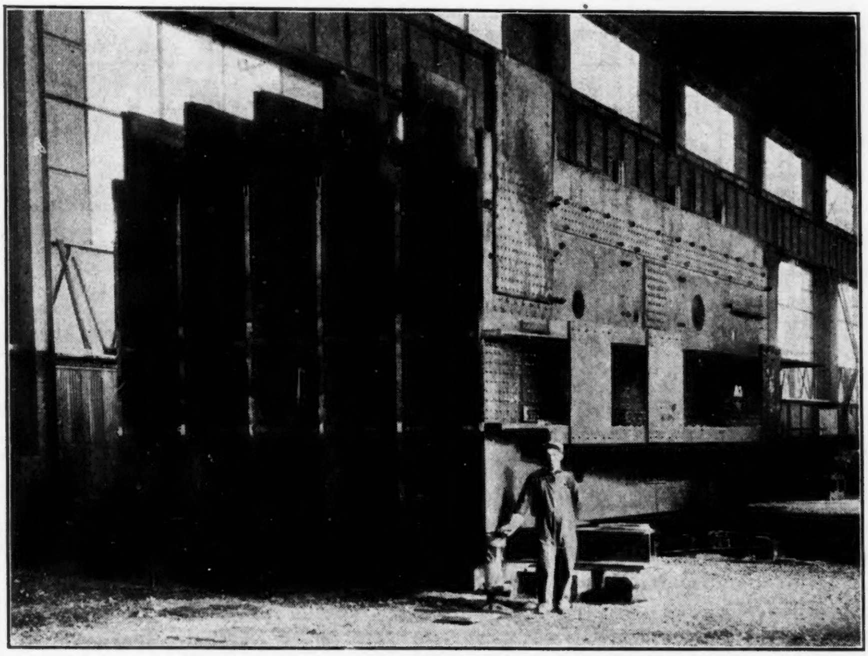
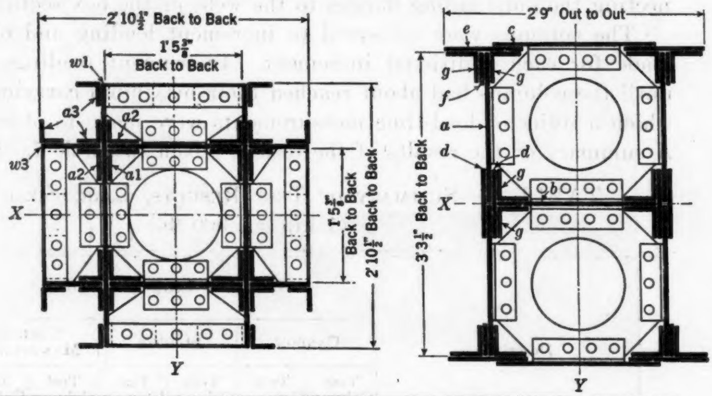


FIG. 18.—VIEW OF TOP SECTION OF INNER COLUMNS.

Eight column sections, of three grades of structural steel, were tested—four with cross-sectional areas of 159 sq. in., two of 155 sq. in., and two of 144 sq. in. Six of these columns were dimensionally half-size specimens of the base columns of the towers of the George Washington Bridge. The remaining two sections were similarly proportioned models of a representative group of the lower chord sections of the Kill van Kull Bridge.

TABLE 6.—CHARACTERISTICS OF TYPICAL TEST COLUMNS



Subject	(a) Column Section, George Washington Bridge				(b) Tower Chord Section, Kill Van Kull Bridge	
	Carbon steel and silicon steel columns		Carbon-manganese steel columns		Carbon-manganese steel columns	
	Symbol		Symbol		Symbol	
Main material	*W* 1	2 plates, 34⅜ in. by ⅝ in.	*W* 1	2 plates, 34⅜ in. by $\frac{9}{16}$ in.	*a*	4 plates, 19¼ in. by $\frac{9}{16}$ in.
	W 2	2 plates, 17 in. by ⅝ in.	*W* 2	2 plates, 17 in. by $\frac{9}{16}$ in.	*b*	1 plate, 20¼ in. by ½ in.
	W 3	4 plates, 7⅝ in. by ⅝ in.	*W* 3	4 plates, 7⅝ in. by $\frac{9}{16}$ in.	*c*	2 plates, 21 in. by ⅜ in.
	a 1	4 angles, 4 in. by 4 in. by $\frac{9}{16}$ in.	*a* 1	4 angles, 4 in. by 4 in. by $\frac{9}{16}$ in.	*d*	4 plates, 9 in. by ½ in.
	a 2	8 angles, 4 in. by 3 in. by ½ in.	*a* 2	8 angles, 4 in. by 3 in. by ½ in.	*e*	4 plates, 10 in. by ⅜ in.
	a 3	12 angles, 3 in. by 3 in. by ½ in.	*a* 3	12 angles, 3 in. by 3 in. by ½ in.	*f*	4 plates, 4 in. by ½ in.
					g	12 angles, 4 in. by 4 in. by ½ in.
Section area, in square inches	....	159	....	151	....	155
Length, in feet	....	24.00	....	24.00	....	20.75
Radius of gyration, in inches	Axis *X–X*	9.97	Axis *X–X*	9.97	Axis *X–X*	14.07
	Axis *Y–Y*	9.97	Axis *Y–Y*	9.97	Axis *Y–Y*	10.51
Slenderness ratio	Axis *X–X*	28.90	Axis *X–X*	28.90	Axis *X–X*	17.70
	Axis *Y–Y*	28.90	Axis *Y–Y*	28.90	Axis *Y–Y*	23.70

The test columns were made of carbon steel, silicon steel, and carbon manganese steel. The material was made to conform to the specifications previously given in this paper. The details of the column sections are shown in Table 6. The column section in Table 6(*a*) was dimensionally one-half the size of the tower column in Panel 0′-1′ of the George Washington Bridge, while that in Table 6(*b*) was one-half the size of the lower chord section, *L*18-*L*19, of the Kill van Kull Bridge.

The total compression in a gauge length of either 20 or 15 ft. was measured; so were the lateral deflection of the column and the possible tendency of the cross-section to change its geometrical shape under test. A secondary study was made of stress distribution at various points of the section by the use of telemeters.

The total compression for the tower columns was measured by sixteen dial compressometers, 20 ft. long, attached four to each column face, two on the edges of the outstanding flanges, and two to the inner corner angles connecting the outstanding flanges to the webs of the box section.

The columns were subjected to increment loading and observations were made for each additional increment. Instrument readings were continued until the columns had about reached their maximum carrying capacity, after which a series of load-time measurements were made to observe any pick-up A summary of the results of the tests is given in Table 7. In this table the

TABLE 7.—SUMMARY OF TEST RESULTS, LARGE-SIZED COLUMNS
(1 kip = 1 000 lb.)

Item	Description	George Washington Bridge						Kild van Kull Bridge	
		Carbon		Silicon		Carbon Manganese		Carbon Manganese	
		Test Column V	Test Column VI	Test Column VII	Test Column VIII	Test Column 1	Test Column 2	Test Column 1	Test Column 2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	Test Data (in Kips per Square Inch):								
1	Proportional limit.........	17.00	18.00	22.00	20.00	24.00	22.00	24.00	23.00
	Stress at Failure:								
2	First maximum.........	33.60	33.50	52.80	53.00	61.56	62.28	59.00	58.65
3	"Pick-up" maximum....	36.80	36.70	55.80	54.90	.....	.....	.....	......
	Yield Point Determined from Mill Test (in Kips per Square Inch):								
4	Average maximum.........	40.60	40.60	57.20	57.20	59.47	59.47	60.48	60.48
5	Average minimum.........	38.70	38.70	56.40	56.40	58.39	58.39	59.74	59.74
6	Weighted maximum.......	41.40	41.40	57.00	57.00	59.75	59.75	60.30	60.30
7	Weighted minimum........	39.00	39.00	52.80	52.80	58.76	58.76	58.86	58.86
8	Percentage efficiency.........	83.6	83.3	96.2	96.5	103.9	105.1	99.0	98.4

values for the proportional limit (Item 1) were determined from the plotted test data showing the relation of applied stress to strain. The first maximum stress at failure (Item 2) is that at which failure is ordinarily considered to occur, in column testing practice. The "pick-up" maximum stress (Item 3) is that which occurs after the point of first maximum stress, following long-continued loading with considerable deformation. The average maximum and minimum yield points (Items 4 and 5) were determined by averaging, respectively, the maximum and minimum yield-point values from coupon tests. The weighted maximum and minimum yield points (Items 6 and 7) were determined by weighing, respectively, the maximum and minimum values from coupon tests on the basis of the ratio of the area of the section (plate or angle) represented by the mill test, to the total section area of the test column. The efficiencies in Item 8, Table 7, are based on the mean of the weighted yield-point values (Items 6 and 7) and the first maximum stress

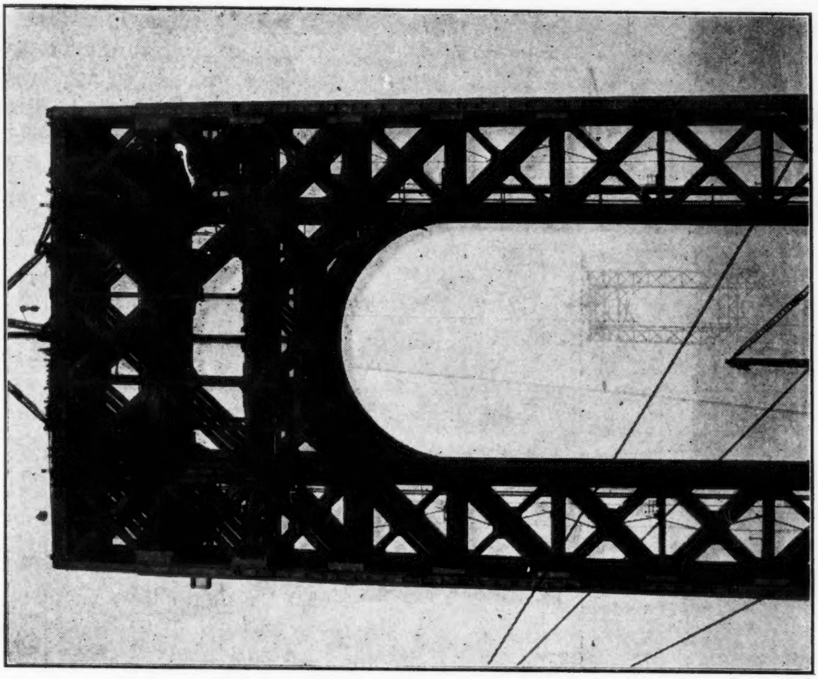


FIG. 20.—VIEW OF UPPER PART OF NEW JERSEY TOWER

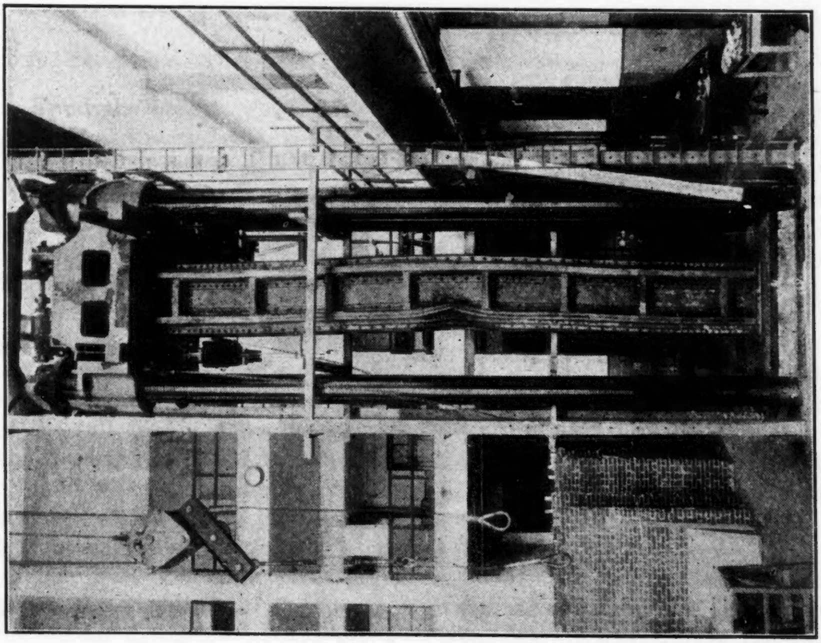


FIG. 19.—TYPICAL FAILURE, A SILICON STEEL TEST COLUMN.

at failure (Item 2). These efficiencies express the ratio of the stress at which the column failed to that of the test coupon. The stress-strain curves developed by the test sections are shown in Fig. 21. The small slenderness ratio of the columns and their box construction did not develop any marked deflections before evidence had developed that the column was at the point of failure. A typical failure of a silicon steel column is shown in Fig. 19.

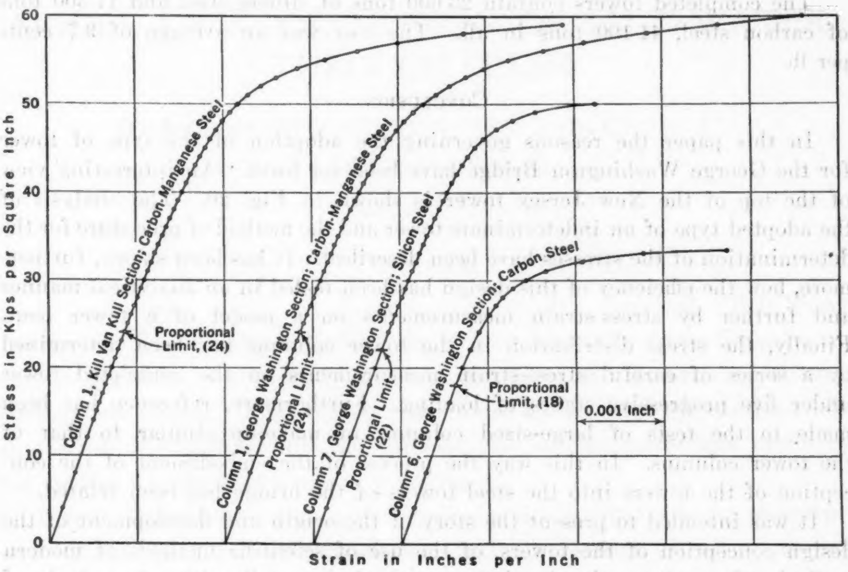


FIG. 21.—STRESS-STRAIN CURVES DEVELOPED BY TEST SECTIONS SHOWN IN TABLE 6.

From the results of the tests it has been found that the ratio of the strength of the test column to the mean average yield point of its material was 93.1 for the first maximum load or 96.6 for the second maximum load. The probable stress, at failure, of the full-sized silicon steel tower column (Panel 0-1; area, 716 sq. in.), based on the foregoing observations is 46 200 lb. per sq. in. for the first maximum load and 47 800 lb. for the second. Since the greatest allowable compressive stress on silicon steel was 23 000 lb. per sq. in., this indicates a factor of safety for the full-sized column of 2.01 to 2.08, respectively.

To interpret the factor of safety correctly it should be kept in mind that the maximum compressive stress of 23 000 lb. is obtained under the reaction of the full dead and live load of the bridge, the dead load being 39 000 lb. and the live load, 8 000 lb. Should the live load be doubled, the stress in the columns will be increased by only 4 000 lb. per sq. in. To reach the ultimate strength of the towers the total load would have to be doubled, which is an evident impossibility.

The results of this investigation on large-sized column sections confirmed the confidence of the engineers in the use of silicon steel and the box section for the tower columns of the bridge. They also furnished the Engi-

neering Profession with information as to the behavior and strength of large fabricated columns and the efficiency of specific types of sections as compressive members. The direct comparison of three grades of structural steel on identical sections has been established by tests.

Tonnage and Cost

The completed towers contain 23 600 tons of silicon steel and 17 500 tons of carbon steel, 41 100 tons in all. The cost was an average of 9.7 cents per lb.

Conclusion

In this paper the reasons governing the adoption of the type of tower for the George Washington Bridge have been set forth. An interesting view of the top of the New Jersey tower is shown in Fig. 20. The analysis of the adopted type of an indeterminate tower and the method of procedure for the determination of the stresses have been described. It has been shown, furthermore, how the efficiency of this design has been tested in an analytical manner and further by stress-strain measurements on a model of a tower bent. Finally, the stress distribution in the tower columns has been determined by a series of careful stress-strain measurements on the completed tower under five progressive stages of loading. Furthermore, reference has been made to the tests of large-sized columns of make-up similar to that of the tower columns. In this way the process of the embodiment of the conception of the towers into the steel towers of the bridge has been related.

It was intended to present the story of the origin and development of the design conception of the towers, of the use of scientific methods of modern analysis of structures in the design, and of the application of methods of critical reasoning to its efficiency and stability. While supported by the engineering experience of the past, it was fully realized that mathematical analysis and reasoning are essentially mental operations and should be verified by observations of the physical behavior, in material embodiment. Observations were then made on a most readily available model of a bent. Finally, an extensive program of stress-strain measurements was carried out on the erected structure to verify the predictions of the engineering assumptions and analysis and to establish the efficiency of the behavior of the towers under load. To verify the behavior and the strength of the most important members of the towers, large-sized columns, about one-fourth the area, were tested to failure. The test results showed that the form adopted for the columns and their material and fabrication met the expectations of the designers.

It has been proved here again that the assumptions and methods of modern structural engineering are in close accord with the processes of Nature. It has given further assurance and confidence that engineering knowledge, the processes of mills and fabricators, and the skill of erectors can well be trusted to produce efficient and safe structures of this, and of greater, magnitude.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

MODEL LAW FOR MOTION OF SALT WATER THROUGH FRESH

BY MORROUGH P. O'BRIEN [1] AND JOHN CHERNO,[2] ASSOCIATE MEMBERS AM. SOC. C. E.

SYNOPSIS

In connection with the investigations of the proposed "Salt-Water Barrier" in San Francisco Bay by the United States Engineer Department, certain questions were raised as to the action of salt water when flowing through fresh water. This problem was studied by means of hydraulic models, in the use of which the dimensional relations between model and prototype for similar flow were of primary importance.

Investigation showed that a model, geometrically similar to its prototype, could not be used for a study of the relative movement of salt water and fresh water, but that it must be distorted so that the scales for vertical dimensions, horizontal dimensions, and salinity satisfy a definite relation, which is expressed by Equations (14) of the paper.

INTRODUCTION

In connection with an investigation of the "salt-water barrier," that has been proposed as a means of keeping sea water from advancing into the delta of the Sacramento and San Joaquin Rivers, in California, certain problems arose regarding the motion of salt water when placed in contact with fresh water. All the sites proposed for the barrier would require locks for vessels of deep draft operating under an average difference in elevation in favor of fresh water of about 3 ft., with a depth over the sill of 40 ft. for the larger locks. At the Dillon Point site, there would be an adverse salt-water head during part of the higher tides. These conditions made it possible for large volumes of salt water to pass through the locks into the fresh-water pool with each inbound lockage and thus defeat the purpose of the barrier. As the

NOTE.—Discussion on this paper will be closed in March, 1933, *Proceedings*.

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available data were scanty and indefinite, and to some extent conflicting, it was decided to study the problem through the use of hydraulic models, and, as the first step, a series of experiments were made in the Hydraulic Laboratory of the University of California to determine the model law governing the motion of salt water through fresh water. Although considerable information was obtained regarding dilution and methods of flushing salt water out of fresh-water reservoirs, this paper is concerned only with the derivation and experimental confirmation of the model law governing the dynamical aspects of the problem.

Notation

The notation used in this paper is adapted to the suggestions of the Society's Special Committee on Irrigation Hydraulics.[3] Symbols that are used only in derivations and do not appear in the final formulas, will be introduced at their proper place and are omitted from the following list:

b = scale ratio.

d = depth of water, in feet; d_h = gate opening.

d' = depth in model; $b_d = \frac{d'}{d}$ = scale ratio for vertical dimensions.

g = acceleration of gravity.

l = width of lock.

$s = G_1 - G_2$ = salinity.

s' = salinity in model; $b_s = \frac{s'}{s}$ = scale ratio for salinity.

t = time, in seconds.

w = 62.5 lb. per cu. ft. = unit weight of water.

A = total area covered by salt water at any time, measured in terms of $V_0\,l$.

G = specific gravity; G_1, of heavy liquid; G_2, of light liquid.

K = model characteristic $= \frac{L_0}{d^{2.5}\,s^{0.5}}$.

L = longitudinal distance along the channel; L_0 = length of lock.

L' = length in model; $b_L = \frac{L'}{L}$ = scale ratio for horizontal dimensions.

V = velocity of salt-water wave front, in feet per second; V_0 = initial velocity of salt-water wave front, in feet per second.

ν = kinetic viscosity as defined on page 1778.

Description of Phenomenon

Before developing the theory or discussing the experimental results, a brief description of the phenomenon observed will be useful in defining the problem. The channel shown in Fig. 1 was arranged so that a vertical gate divided it into two compartments. The smaller compartment was filled with colored salt water while the remainder of the channel was filled with fresh water. The water surfaces on the two sides of the gate were brought to the same elevation, and the gate was then removed suddenly. The heavier salt water

[3] *Proceedings*, Am. Soc. C. E., May 1932, p. 729.

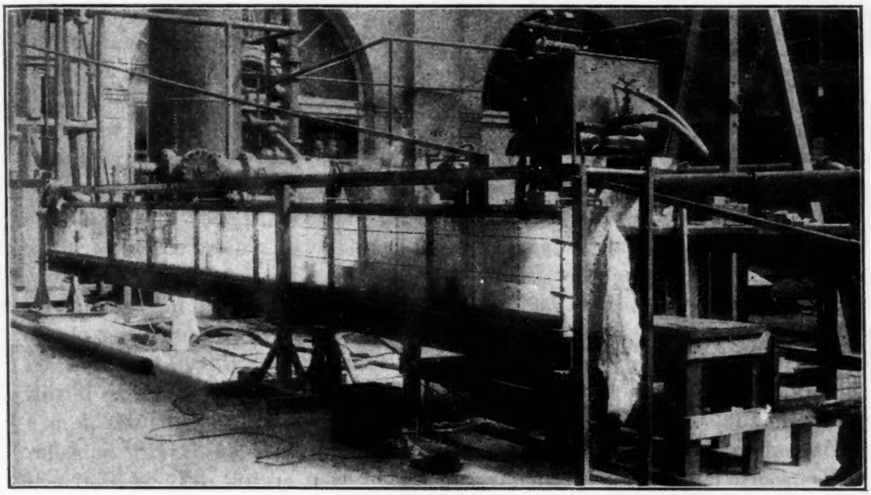
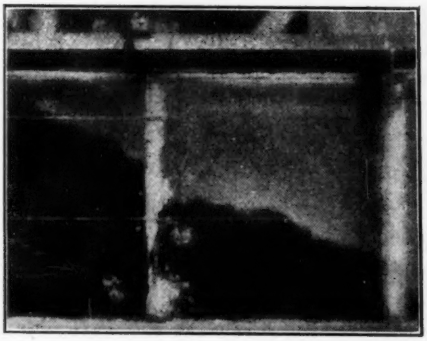


FIG. 1.—VIEW OF EXPERIMENTAL CHANNEL WITH GLASS SIDES.



FIB. 2.—SALT WAVE LEAVING LOCK.

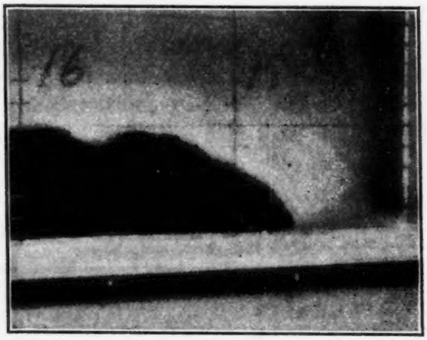


FIG. 3.—SALT WAVE AT ABOUT EIGHT LOCK LENGTHS FROM GATE.

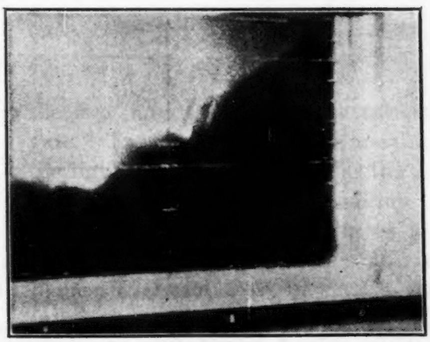


FIG. 4.—SALT WAVE STRIKING BULKHEAD END OF CHANNEL.

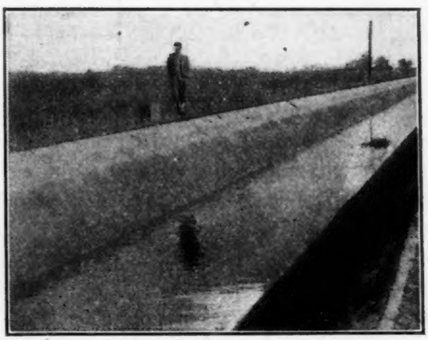
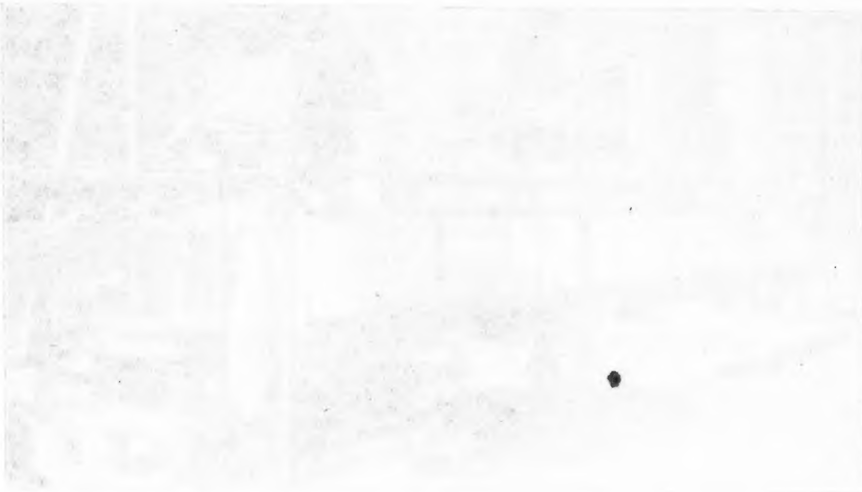


FIG. 5.—EXPERIMENTAL CHANNEL AT TURLOCK, CALIF.



slid under the fresh water and moved along the bottom of the channel with a peculiar tumbling motion while the fresh water moved in the opposite direction along the surface.

The condition immediately after the gate was removed is shown in Fig. 2, the white streak in the middle of the photograph marking the original position of the gate. Fig. 3 shows the advancing wave front, which for sea water (specific gravity = 1.026), with a depth of 1.2 ft., had a velocity of approximately 0.50 ft. per sec. At some instant subsequent to the removal of the gate, the fresh water, moving into the smaller chamber, reached the rear wall, and the salt water continued to move along the channel as a slug of constantly diminishing height and increasing length. The initial height of the salt-water wave was almost exactly one-half the total depth as appears in Fig. 2. In Fig. 3, showing the wave at eight lock lengths from the initial position, the height has decreased to four-tenths of the depth. In Fig. 4, the advancing wave has piled up against the end of the channel and a reflected wave is about to start back along the surface of separation of the two liquids. Fig. 5 is a view of an experimental channel at Turlock, Calif., discussed subsequently.

THEORY

Consider a channel of rectangular cross-section and horizontal bottom, divided into two compartments containing liquids of different specific gravities. It is evident that, when the two liquid surfaces are at the same elevation, the forces on the two sides of the partition are not equal and that a relative motion of the two liquids will result when the partition is removed. Referring to Fig. 6, the line, *A O B*, represents the surface of separation of

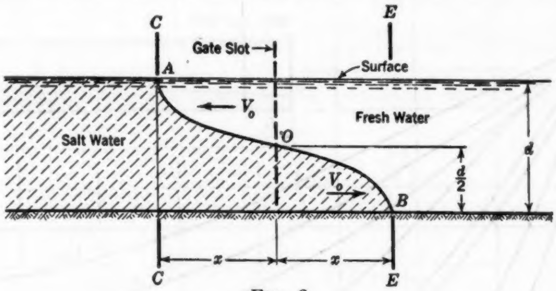


FIG. 6.

the two liquids at some time subsequent to an instantaneous removal of the gate. The heavier liquid is moving along the bottom, displacing a part of the lighter liquid, which is moving in the opposite direction along the surface.

Observation shows no motion of the salt water to the left of Section *C-C* and of the fresh water to the right of Section *E-E*, but all parts of both liquids acquire their full velocities, when they are reached by the planes, *C-C* and *E-E*, in such a short time that, for all practical purposes, infinite acceleration can be assumed. It was further observed that the point, *O*, is at one-half the depth and that no change in surface elevation occurs, so that the quantities of fresh water and salt water passing the gate section

are equal and move with equal velocity, $V_0 = \frac{dx}{dt}$. The unbalanced force acting toward the right on the mass, $M = 2 \times \frac{d}{2} G_1 \frac{w}{g} \times 2 \times \frac{d}{2} G_2 \frac{w}{g}$, of water between Sections *C-C* and *E-E* is;

$$w G_1 \frac{d^2}{2} - w G_2 \frac{d^2}{2} = w s \frac{d^2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and is equal to the change of the momentum, $M V_0$, per second. Therefore,

$$w s \frac{d^2}{2} = \frac{d}{dt} (M V_0) = \frac{w}{g} d (G_1 + G_2) \frac{d}{dt}\left(x \frac{dx}{dt}\right) \ldots\ldots\ldots (2)$$

The first integral, considering that $x = 0$ for the moment, $t = 0$, when the gate is removed, is,

$$s \frac{d}{2} t = \frac{G_1 + G_2}{g} x \frac{dx}{dt} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The second integral is, therefore,

$$x = \pm t \sqrt{\frac{g d s}{2 (G_1 + G_2)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Therefore,

$$V_0 = \frac{dx}{dt} = \pm \sqrt{\frac{g d s}{2 (G_1 + G_2)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Since V_0 is independent of x and t, it follows that all salt water and fresh water is brought instantly to an initial velocity, V_0, until either Section *C-C*

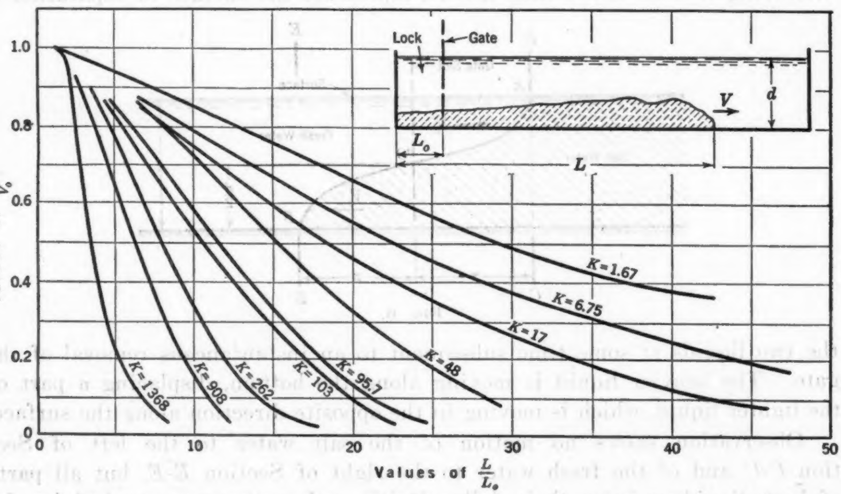


FIG. 7.—RELATION BETWEEN RATE OF DECREASE OF VELOCITY AND MODEL CHARACTERISTIC *K* (LABORATORY EXPERIMENTS).

or Section *E-E* reaches the end of the testing flume. After that the foregoing assumed conditions no longer exist, and the subsequent advance of the salt water takes place, as shown in Fig. 7.

Tests might be expected to show initial velocities slightly less than V_0 as computed by Equation (5) because no friction losses have been taken into account. However, it will be shown presently that delays in opening the gate will cause initial velocities larger than V_0, so that no prediction of a systematic deviation can be made.

Another method of obtaining Equation (5) is to consider the potential energy converted into kinetic energy during the downward motion of the heavier liquid. If, in Fig. 8, d_g is the vertical distance from the sill of

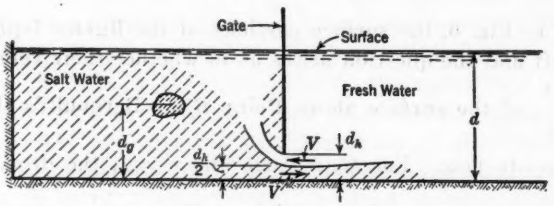


FIG. 8.

the gate to the center of gravity of a volume of liquid, B, which ultimately passes through the gate, the velocity of efflux can be obtained on the assumption that a counter flow of the lighter liquid occurs with the surface of separation at $\frac{d_h}{2}$, and with a velocity equal to the outflow velocity of the salt water. This is equivalent to assuming that the surfaces on the two sides of the gate remain at the same elevation after the gate is opened, and is based on direct observation. The center of gravity of B falls through the distance, $d_g - \frac{d_h}{4}$, but an equal volume of the lighter liquid is raised through the same distance. The decrease in the potential energy of the system is, then,

$$E_d = B\,w\,s\left(d_g - \frac{d_h}{4}\right)$$

The kinetic energy is,

$$E_K = \frac{B\,w\,V_1^2}{2\,g}\,(G_1 + G_2)$$

and, if the slight frictional losses are neglected,

$$\left(d_g - \frac{d_h}{4}\right) s = \frac{V_1^2}{2\,g}\,(G_1 + G_2) \quad \text{.................(6)}$$

If the lock is filled with salt water and the gate is opened a small distance, the initial velocity is obtained by making $d_g = d$ and $d_h = 0$, since the first fresh-water particles passing through the gate rise from the bottom to the surface. The initial velocity of efflux is, then,

$$V_1 = \sqrt{\frac{2\,g\,d\,s}{G_1 + G_2}} = 2\,V_0 \quad \text{.........................(7)}$$

Equation (7) reduces to the ordinary orifice equation when $G_2 = 0$, corresponding to the discharge of water into air. If the gate is removed, $d_g = \frac{d}{2}$, $d_h = d$, and V_1 becomes equal to V_0 as obtained from Equation (5) for the same conditions (that is, removal of the gate).

Since a finite time is required for raising the gate, it follows that the observed initial velocity will lie between the value given by Equations (5) and (7), approaching the former value the more closely, the shorter the time of opening.

Referring to Fig. 6, the surface particles of the lighter liquid are moving toward the left and the question arises as to whether their velocity is created by a drop, $\frac{V_0^2}{2g}$, of the surface above Point B, as Bernoulli's theorem would require for steady flow. For $G_d = 0.025$ and $d = 1$, the drop, $\frac{V_0^2}{2g}$, would be about 0.003 ft. and, therefore, visible. The only surface disturbances observed, however, were those directly attributable to the opening of the gate and these were much smaller than the aforementioned amount.

As the total volume of liquid in the testing flume is constant, a drop of the surface at any place would have to be made up by a rise of the surface at some other place; that is, water would have to be lifted from below the original surface to above it, and, therefore, no drop of the surface could furnish the energy required to set water in motion. On the contrary, it would diminish the kinetic energy. Although it is not the intention to investigate herein the mechanism by which the different parts of the two liquids are set in motion, it can be stated that vertical displacements of the surface are not only unnecessary for the creation of the observed movements, but are opposed to them. It follows from the derivation of Equation (6) that such displacements would be reflected in a deficiency of V_0 and as shown subsequently, if such a deficiency exists at all, it is negligible.

The Model Law

The term, "model," refers to any hydraulic system by means of which the behavior of another system called the prototype, can be predicted, and the term, "model law," refers to the method of such prediction. The model law is a set of rules which assigns to each point of the prototype a corresponding point in the model and assigns the relations existing between the physical properties at corresponding points.

In the usual case, the required point-to-point relationship between the model and its prototype is obtained by making them geometrically similar at least to the extent of using only one scale ratio for horizontal dimensions and one for vertical dimensions. When these scale ratios are equal, the model and prototype are geometrically similar in the ordinary sense; otherwise, the model is "distorted." Another requirement is that the ratio of the measured quantities at pairs of corresponding points shall be the same for all points. For example, if the velocity in the model is one-half the velocity

in the prototype at any pair of corresponding points, the velocity at any point in the model should be one-half the velocity at the corresponding point in the prototype. In other words, the scale ratio of velocities is to be a constant and a similar relation should hold true of all other measured quantities.

Whether it is possible to build models that represent their prototypes in every detail will not be discussed here. In practice, models are always used for the study of a limited number of phenomena and should be built so that these phenomena are represented properly. Minor associated phenomena must often be represented incompletely or even incorrectly and the success of a model experiment depends upon whether these neglected phenomena are controlling factors.

The present problem is the derivation of the model law governing the intrusion of salt water into fresh water in a shallow basin such as San Francisco Bay. The ultimate problem is a study of the distance traversed by the salt water, the dilution it undergoes during the motion, and its velocity at points along its path. It is known from direct observation that the vertical motion involved can be neglected because it is small as compared with the horizontal motion and because it occurs in a time that is short as compared with the time required for the entire process, and the model law will be based upon these assumptions.

Transference of the data from the model to the prototype will require the six ratios: Horizontal dimensions; vertical dimensions; velocities; time; salinity; and dilution. The dilution itself is a ratio and has no dimensions and, consequently, its value should be the same in model and prototype at corresponding points. The first five scale ratios are not independent and it is necessary to determine the relation between them.

In the prototype, the velocity is, $V = \frac{dx}{dt}$, and the corresponding quantity in the model is, $V' = \frac{dx'}{dt'} = \frac{b_L}{b_t}\frac{dx}{dt} = \frac{b_L}{b_t} V$. The velocity ratio is, therefore,

$$\frac{V'}{V} = b_v = \frac{b_L}{b_t} \quad \text{.........................(8)}$$

From Equation (5), the initial velocities are,

$$V_0 = \sqrt{d' s} \sqrt{\frac{g}{2(G_1 + G_2)}}$$

in the prototype, and,

$$V'_0 = \sqrt{d' s'} \sqrt{\frac{g}{2(G'_1 + G'_2)}}$$

in the model. Therefore,

$$\left(\frac{V'_0}{V_0}\right)^2 = \frac{d'}{d} \times \frac{s'}{s} \times \frac{G_1 + G_2}{G'_1 + G'_2}$$

or,

$$b^2_v = b_d\, b_s\, b^{-1}_{(G_1 + G_2)} \quad \text{.........................(9)}$$

If salt water and fresh water are to be used in the model, $G_2 = 1.00$, and G_1 ranges from 1.00 to 1.20, so that $b_{(G_1 + G_2)}$ ranges from about 0.9 to 1.1 and ordinarily will be close to 1.0. For simplicity this ratio will be assumed equal to 1.00 in the following discussion unless otherwise specified. On this basis, Equation (9) becomes,

$$b^2_v = b_d\, b_s \qquad (10)$$

A third relation between the scale ratios is found in the general equations of internal friction, which have the form,

$$\frac{w}{g}\frac{dV}{dt} + \frac{dp}{dx} - \mu \left\{ \frac{d^2V}{dx^2} + \frac{d^2V}{dy^2} + \frac{d^2V}{dz^2} \right\} = 0$$

for the X-direction, which is horizontal. The first term represents the inertia of a particle; the second, the unbalanced force acting on it in the X-direction; and, the third, the internal friction. Here, μ is a coefficient of internal friction which is ordinarily the coefficient of absolute viscosity. In the present case, μ may differ from the viscosity of either of the liquids in contact. The absolute viscosity divided by the density, $\frac{w}{g}$, is usually referred to as the kinematic viscosity and will be represented by ν. It will be assumed to have the same value for both model and prototype unless otherwise specified.

A similar equation can be written for the Y-direction which is also horizontal. The Z-direction is not considered since vertical motions are neglected.

Let P denote a hydraulic force. Fig. 9 shows that $\frac{dP}{dx}$ may be written as $\frac{d\,(w\,H\,s)}{dx}$ because of the horizontal water surface. Direct observation shows that salt water tends to move along the bottom of a channel as a coherent slug and that fresh water moves over the top of it at about the same velocity

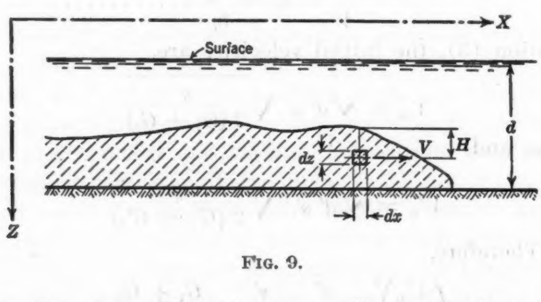


FIG. 9.

in the opposite direction. The velocity gradient, therefore, is large at the surface of separation and relatively small in all other directions and at all other positions. In other words, practically all the work of internal friction is done in the transition layer.

In view of the foregoing considerations, the equation of motion is simplified to,

$$\frac{dV}{dt} + g\frac{d(H\,s)}{dx} - \nu\frac{d^2V}{d\,z^2} = 0 \quad \ldots\ldots\ldots\ldots\ldots(11)$$

after dividing through by $\frac{w}{g}$. Writing the corresponding equation for the model and remembering that H is a vertical dimension,

$$\frac{d\,V'}{dt'} + g\frac{d(H's')}{dx'} - \nu\frac{d^2\,V'}{d\,z'^2} = \frac{b_v}{b_t}\frac{d\,V}{dt} + \frac{b_d\,b_s}{b_L}\frac{d(H\,s)}{dx} - \frac{b_v}{b^2_d}\nu\frac{d^2V}{dz^2} = 0 \quad ..(12)$$

Equations (11) and (12) can be equal only if,

$$\frac{b_v}{b_t} = \frac{b_d\;b_s}{b_L} = \frac{b_v}{b^2_d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(13)$$

The physical meaning of Equation (13) is that the inertia, pressure, and friction forces in the prototype must be reduced in the same ratio in the model.

Equations (8), (10), and (13) are to be satisfied simultaneously. Taking b_d and b_s as the independent variables, these equations give as the model relations:

$$b_L = b_d^{2.5}\,b_s^{0.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(14a)$$

$$b_v = b_d^{0.5}\,b_s^{0.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(14b)$$

$$b_t = b_d^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(14c)$$

The ratios, b_s and b_d, have been chosen as the independent variables because they are easiest to control in practice. Equation (14*a*) is of primary importance in the construction of a model. Replacing the ratios by characteristic dimensions such as the length of a lock, L_o, the depth, d, and the initial salinity, s, Equation (13) gives,

$$\frac{b_L}{b_d^{2.5}\,b_s^{0.5}} = 1 = \frac{L'_o}{L_o}\left(\frac{s}{s'}\right)^{0.5}\left(\frac{d}{d'}\right)^{2.5}$$

Separating the quantities for model and prototype,

$$\frac{L'_o}{d'^{\,2.5}\,s'^{0.5}} = \frac{L_o}{d^{2.5}\,s^{0.5}} = K \quad \ldots\ldots\ldots\ldots\ldots\ldots(15)$$

in which, K is the simplified criterion of similarity.

Retaining both g and ν as variables, the complete model law is,

$$\frac{L_o\,\nu}{g^{0.5}\,s^{0.5}\,d^{2.5}} = K' \quad \ldots\ldots\ldots\ldots\ldots\ldots(16)$$

which is dimensionless. Equation (15) is applicable to the case of salt water and fresh water because ν is approximately constant, but Equation (16) may be necessary if liquids are used in the model entirely different from those in the prototype. The ratio, $b_{(G_1+G_2)}$, might also have to be included in this case.

Experimental Procedure

The glass-sided channel (Fig. 1) used in the first series of experiments, was 24.4 ft. long, 0.5 ft. wide, and 1.25 ft. deep, placed so that the bottom was horizontal. A thin, sheet-metal gate, moving in vertical guides placed above the channel proper and sealed with rubber strips, coated with grease, was used instead of a lock-gate. The volume displaced by the gate was insignificant, and the surface distuıbances resulting from its removal were not sufficient to have an appreciable effect on the velocities. The specific gravities of the liquids were measured by a hydrometer or a specific gravity balance, and time intervals by a chronograph and stop-watches. A co-ordinate system in feet and tenths was drawn on the glass plates with a grease pencil.

The liquid used to fill the lock was a solution of crude granulated rock salt in water, colored to a deep red with potassium permanganate. As the gate was unable to withstand any considerable side thrust, the channel was first filled to the desired depth with fresh water, and the gate was then put in position and sealed with cup grease. The fresh water was siphoned from the surface of the lock chamber as the saline solution was discharged at the bottom, and the two water surfaces were brought to the same elevation by point-gauges reading to 0.001 ft. The lock chamber was then thoroughly stirred by bubbling before the specific gravity was measured. The behavior of the salt-water following the removal of the gate has been described previously.

Following the laboratory experiments, a few additional tests were made in a concrete-lined irrigation canal of trapezoidal section at Turlock, Calif. (See Fig. 5.) This channel had a bottom width of 6 ft., side slopes of 1 on $1\frac{1}{2}$, a maximum depth of 4 ft., a bottom slope of 1 on 2 000, and a length of 2 600 ft. The gate, which consisted of a trapezoidal wooden frame covered with oil-cloth, was placed at the lower end of the experimental section so that the advancing wave traveled up along the bottom of the channel. The manner of conducting the tests was similar to that in the laboratory, but the probable error was much greater. In addition, a current along the channel, due to leakage past the gates, was noticeable in some experiments. During certain experiments, wind-driven surface currents created counter currents on the bottom opposing the progress of the salt solution. The gate was lifted by block and tackle and the time required to lift it clear of the water surface was relatively greater than in the laboratory, a condition which, according to Equation (7), would result in a relatively higher initial velocity. The channel and gate used at Turlock appears in Figs. 10 and 11.

Agreement Between Theory and Experiment

In Table 1, the measured initial velocities in the laboratory experiments are compared with the velocities computed from Equation (5). The data were picked so as to give as a wide range of d, s, and L_0 as possible and to include tests giving the maximum deviation from the formula. The differences between the experimental results and the equation are erratic and do not indicate any general tendency for the actual velocity to be greater or less

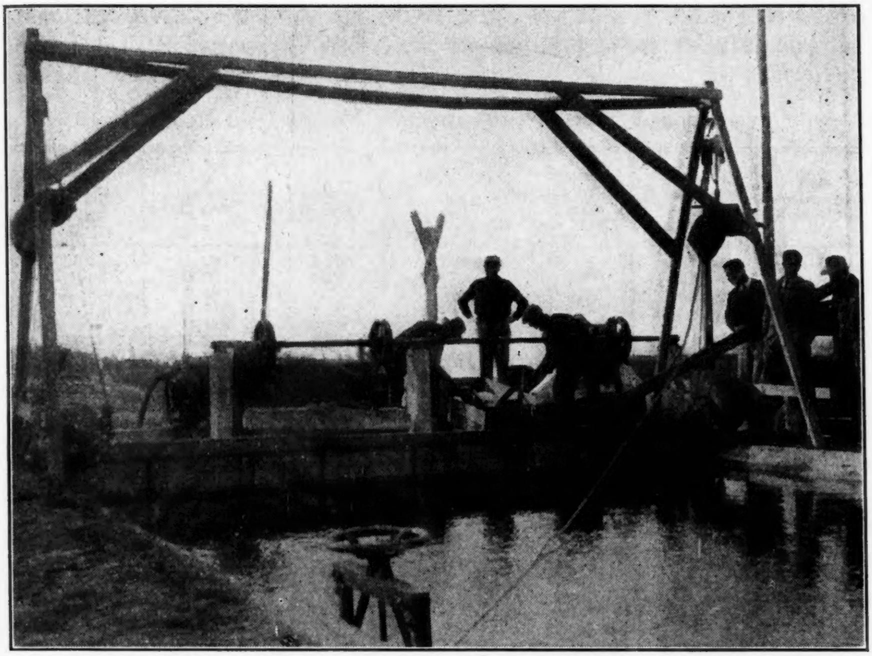


Fig. 10.—Turlock Experiments: Gate Lowered for Start of Experiment.

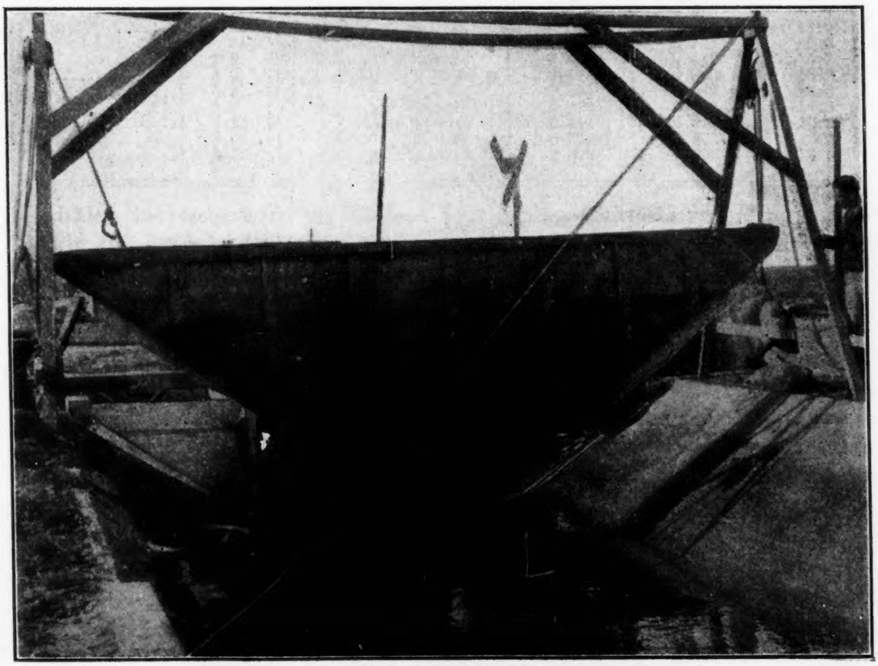


Fig. 11.—Turlock Experiments: Gate Raised.

than that given by the formula. Considering the data of Table 1, it can be stated that Equation (5) represents the initial velocity in a rectangular channel with a probable error of 2 per cent.

TABLE 1.—INITIAL VELOCITIES MEASURED IN LABORATORY

Test No.	d, in feet	L_O, in feet	s	V_O, in feet per second (computed)	V_O, in feet per second (observed)	Percentage, deviation
A-2	1.20	1.79	0.029	0.528	0.595	+11.2
A-5	0.80	1.79	0.030	0.437	0.416	—4.8
A-6	0.40	1.79	0.032	0.319	0.298	—6.3
A-25	0.415	0.90	0.0302	0.316	0.297	—6.0
A-23	0.412	0.32	0.0319	0.324	0.298	—8.0
A-14	1.20	8.95	0.032	0.554	0.532	—4.0
A-10	1.20	0.39	0.023	0.470	0.436	—7.1
A-4	1.20	1.79	0.015	0.380	0.379	—0.2
A-12	1.19	1.79	0.063	0.767	0.912	+18.9
A-13	1.20	1.79	0.101	0.970	0.995	+ 2.6

Table 2 shows the initial velocities measured in the irrigation channel at Turlock. As would be expected from the time required to open the gate, the observed velocities are in excess of the computed values, and the percentage

TABLE 2.—INITIAL VELOCITIES MEASURED IN TURLOCK, CALIFORNIA

Test No.	d_O	L_O	G_O	V_O (Computed)	V_O (Observed)	Error
T-1	2.1	10.5	0.0091	0.392	0.374	-4.6
T-2	3.1	10.5	0.0110	0.523	0.578	10.1
T-3	3.95	10.5	0.0113	0.598	0.693	15.9
T-4	3.85	20.6	0.0110	0.582	0.735	26.2
T-5	1.8	20.2	0.0123	0.421	9.445	5.7
T-6	3.0	20.3	0.0100	0.490	0.540	10.2
T-7	3.8	5.4	0.0083	0.503	0.647	28.5
T-8	2.9	5.2	0.0095	0.470	0.536	14.0
T-9	2.25	5.7	0.0094	0.411	0.459	10.8
T-10	3.05	6.4	0.0234	0.752	0.873	16.2
T-11	4.00	6.4	0.0215	0.828	0.893	7.9

deviation increases with the depth. Segregating the tests according to the depth, the average deviations for depths of 2, 3, and 4 ft. are found to be 4.0%, 12.6%, and 19.6%, respectively.

SPREADING EXPERIMENTS

An additional check on the model law was made in a sheet-metal tank of irregular shape, approximately 25 ft. long, 6 ft. wide, and 1 ft. deep. The method of conducting the experiments was entirely similar to that in the glass channel with the difference that the bottom was covered with a thin layer of sand, on which a co-ordinate system of white lines was drawn with plaster of Paris. A lock, 1.04 ft. long and 0.82 ft. wide, with wing-walls, forming a continuation of the sides of the lock and projecting 0.4 ft., was placed at the side of the tank for part of the tests and at the end for the remainder. This arrangement permitted lateral motion of the salt solution which had not been considered in any of the previous experiments. In deriving Equation (12) only the forces in the X-direction were considered; a simi-

lar line of reasoning applied in the Y-direction would yield the same result, and, consequently, the same model characteristic, K, should completely describe the motion in all systems having horizontal dimensions proportional to L_0 and depth proportional to d.

The results of these experiments on the effect of the spreading are shown in Fig. 12, for the lock at the end of the tank (Fig. 12(a)), and the side,

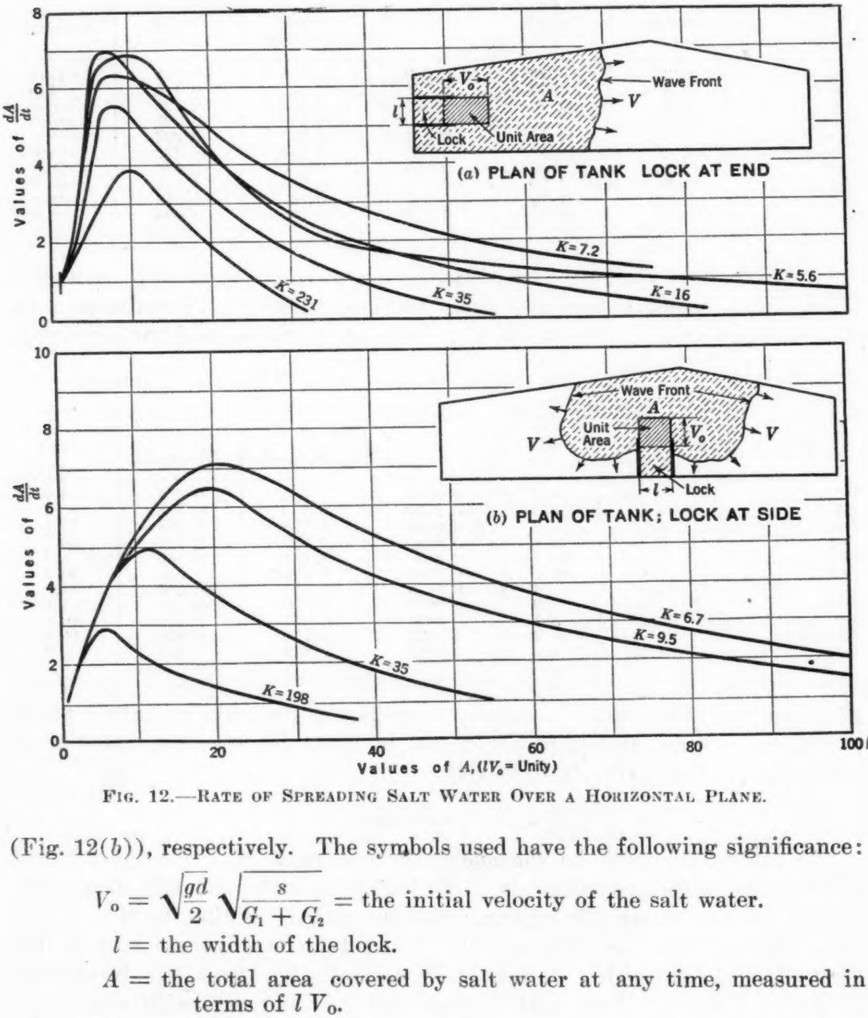


FIG. 12.—RATE OF SPREADING SALT WATER OVER A HORIZONTAL PLANE.

(Fig. 12(b)), respectively. The symbols used have the following significance:

$V_0 = \sqrt{\frac{gd}{2}} \sqrt{\frac{s}{G_1 + G_2}}$ = the initial velocity of the salt water.

l = the width of the lock.

A = the total area covered by salt water at any time, measured in terms of $l\,V_0$.

$\frac{dA}{dt}$ = the time rate of increase of A.

As explained previously, the bottom of the channel was covered with a co-ordinate system of white lines, and as the wave spread out from the lock in all directions, the time at which the wave front reached various points in

that system was noted. Contours representing the area covered at any time were then drawn and, by measuring the areas inside these lines, the values of A and $\frac{dA}{dt}$ were obtained.

In the initial stage of the motion, the velocity is relatively great and the time is difficult to measure precisely, so that the experimental points show considerable scattering in this region; but as the velocity decreases, the points fall more regularly. All the curves exhibit the same general shape and, with only one exception, they arrange themselves according to the value of K. The length of the lock was constant for all these experiments, but the depth ranged from 0.18 to 0.8 ft. and the specific gravity of the salt solution ranged from 1.046 to 1.098.

The model law derived previously indicated that when the value of $\frac{L_0}{d^{2.5}\ s^{0.5}}$ is the same for two structures having horizontal dimensions proportional to L_0 and vertical dimensions proportional to d, the entire motion of the salt water should be similar. A few of the tests made in the glass channel are shown in Fig. 7. The ordinates represent the velocity in percentage of the initial velocity of advance. The abscissas represent the number of lock lengths from the closed end of the lock to the point at which the velocity was measured. It is to be noted particularly that all the curves have the same general shape and that they distribute themselves according to the value of K. Twenty-three curves similar to those in Fig. 7 were used to obtain Fig. 13,

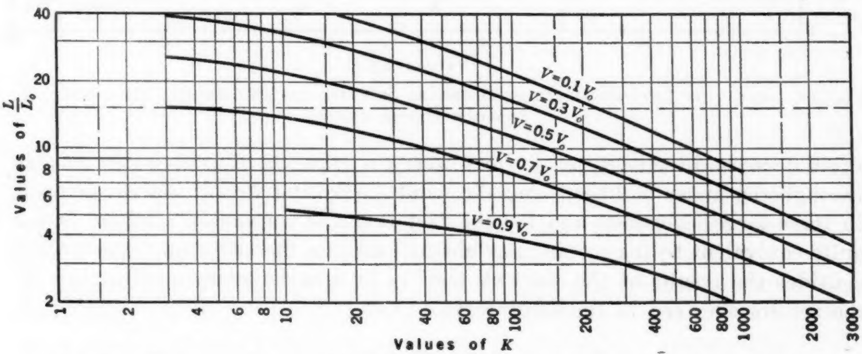


FIG. 13.—RELATION BETWEEN RATE OF DECREASE AND MODEL CHARACTERISTIC K.

which shows the relation between the number of lock lengths from the closed end of the lock, at which the velocity is reduced to a certain percentage of the initial velocity, and K, the model characteristic.

The curves of Fig. 13 permit drawing the velocity curve for any value of K. The actual velocity in any particular case may be obtained by multiplying each ordinate by the initial velocity given by Equation (5).

Fig. 14 shows the velocity curves obtained at Turlock. As mentioned previously, the initial velocity is in most cases high, but the curves for all twelve tests arranged themselves according to the value of K. Only six of

the curves are shown on the graph, in order to avoid confusion, but the remainder showed equal regularity both in shape and location.

In Fig. 14, curves (dotted) drawn from the data in Fig. 13 are compared with the actual data obtained at Turlock for equal values of K, the model characteristic. The initial velocity at Turlock was greater than that observed in the laboratory, but the velocity diminished more rapidly, due principally to the upward slope of the bottom. Other contributing factors in the more

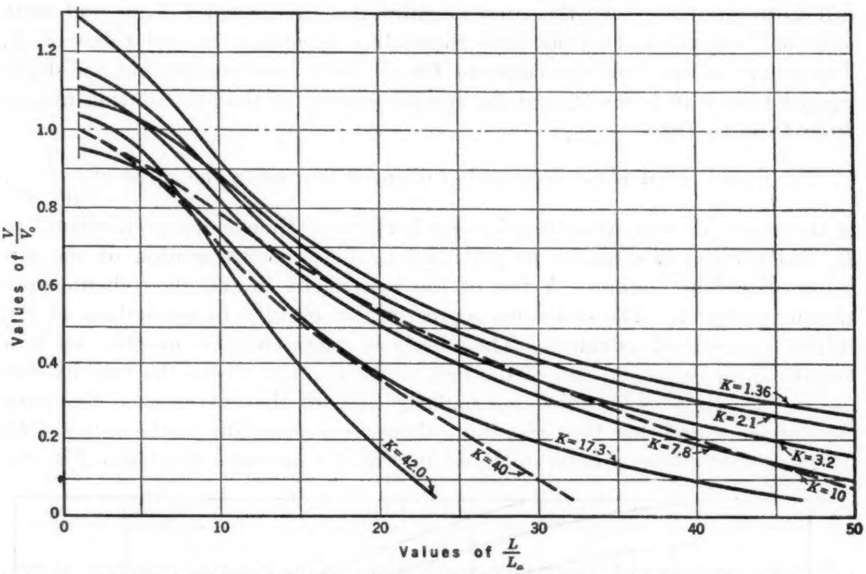


FIG. 14.—RELATION BETWEEN RATE OF DECREASE OF VELOCITY AND MODEL CHARACTERISTIC K; TURLOCK EXPERIMENTS.

rapid decrease in velocity at Turlock were the currents of fresh-water leakage through the upper bulkhead, and the wind. Allowing for all sources of error in these large-scale tests and for the fact that the trapezoidal channel is not quite equivalent to the rectangular channel used in the laboratory, the writers consider the results of the Turlock tests to be a satisfactory confirmation of the results obtained in the laboratory.

CONCLUSIONS

From an assumed character of the forces involved in the motion of salt water through fresh water, the model law governing the similarity was determined to be $\frac{L_0}{d^{2.5}\ s^{0.5}} = \text{constant} = K$, in which, L_0 is any characteristic horizontal dimension, d is a characteristic vertical dimension, and s is a characteristic salinity.

The experimental results show that, regardless of the absolute values involved, the velocity curves distribute themselves according to the values of K, and for two tests having values of K that are approximately equal, the

curves lie very nearly parallel throughout their length. From this, it may be concluded that, if the difference in the values of K for two tests is zero, the velocity curves will coincide, and, consequently, K is the criterion for dynamical similarity.

Furthermore, since the model law, as derived theoretically, coincides with that found experimentally, the fundamental assumption made in the derivation of the law — that is, the assumption that the internal friction acts like a force equal to $\nu \frac{d^2 V}{dz^2} dx\, dy\, dg$ — is essentially correct and no forces other than the two considered in deriving the model law are of major importance.

One other force to which attention was paid is the friction at the bottom of the testing flume. Most of the tests were run on a hardwood bottom painted with red lead. Similar tests were run with the bottom covered with coarse sand for comparison, but no difference in the velocity of the salt water was found. Since the fresh water moves in a direction opposite to that of the salt water in the upper part of the flume, the vertical velocity gradient is exceedingly large at the surface of separation of the two liquids, and it is natural that the friction developed in these transition layers should be great enough to govern the motion almost completely.

It is to be emphasized that Equation (15) is valid only for cases in which the principal motion is horizontal. It is also restricted to experiments in which the kinematic viscosity is the same in both model and prototype; otherwise, Equation (16) is to be used. Another restriction is that the velocities and depths in the model must not produce laminar flow, if turbulent flow exists in the prototype.

The present experiments do not indicate any quantitative limitations of the validity of Equation (15). However, the writers are of the opinion that an actual lock having a length twenty to thirty times its depth could be studied in a model having a length only one-half, or even one-third, of the depth using salt water in the model, but they do not feel that the present experiments alone justify the use of such a combination as ether and mercury to represent the fresh and salt water, respectively. Further experimental work should be done before such a drastic step is taken.

As an illustration of the application of these conclusions, assume that model tests are to be made concerning the distribution and dilution of salt water flowing from the Ballard Locks into the Lake Washington Ship Canal at Seattle, Wash., and compute the dimensions to be used in the construction of a model according to the model law derived in this paper. The actual dimensions are: Length of canal from locks to Lake Washington, 33 000 ft.; depth, length, and width of the locks, 37, 825, and 80 ft. respectively. The law gives a relation between three scale ratios, namely, a ratio for vertical dimensions, b_d; one for horizontal dimensions, b_L; and one for salinity, b_s. Two of the scale ratios can be chosen arbitrarily and the third one is then determined by the relation,

$$\frac{b_L}{b_d^{2.5}\ b_s^{0.5}} = 1 \qquad (17)$$

Table 3 shows the dimensions assumed by the large lock, and by the distance from the locks to the entrance into Lake Washington in models of different scales. A set of vertical scales is chosen, and it is assumed that either the tests would be run with water of the same salinity as that of Puget Sound (which is about 0.02), giving a salinity ratio of unity, or with water of a salinity of 0.2 which is about as high as could be made, and in which case the salinity ratio would be 10:1. The horizontal scales as computed from the model law and the resulting dimensions of the large lock and the distance to Lake Washington in the model, are also shown in Table 3.

TABLE 3.—ILLUSTRATIVE EXAMPLE; DISTRIBUTION AND DILUTION OF SALT WATER INTO LAKE WASHINGTON SHIP CANAL

Vertical scale, b_d	Salinity Scale; 1:1					Salinity Scale; 10:1				
	Horizontal scale, b_L	Distance to Lake Washington, in feet	Lock Dimensions			Horizontal scale, b_L	Distance to Lake Washington, in feet	Lock Dimensions		
			Depth, in feet	Length, in feet	Width, in feet			Depth, in feet	Length, in feet	Width, in feet
1:2	1:5.6	5 900	18.5	148	14.3	1:1.77	18 700	18.5	466	45
1:5	1:55.9	591	7.4	14.8	1.43	1:17.7	1 870	7.4	46.6	4.5
1:10	1:316	105	3.7	2.6	0.25	1:100	330	3.7	8.25	0.8
1:20	1:1770	18.7	1.85	0.47	0.05	1:560	59	1.85	1.48	0.14

ACKNOWLEDGMENTS

These studies were made by the United States Engineer Department, San Francisco District, in conjunction with the University of California. They formed a part of investigations made for a "Report on Proposed Salt Water Barrier, Sacramento, San Joaquin, and Kern Rivers, California," pursuant to the provisions of House Document 308, 69th Congress, First Session.

The investigations were made under the direction of E. H. Ropes, Maj., Corps of Engineers, U. S. A., Assoc. M. Am. Soc. C. E., District Engineer; C. A. Mees, M. Am. Soc. C. E., Principal Engineer, who was in charge of the work with H. G. Gerdes, Assoc. M. Am. Soc. C. E., Associate Engineer as Assistant. Most of the testing was done in the Hydraulic Laboratory of the University of California and the remainder in an irrigation ditch of the Turlock Irrigation District. The writers were in charge of the tests, Professor O'Brien for the University of California and Mr. Cherno for the U. S. Engineer Department.

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DISCUSSIONS

STEREO-TOPOGRAPHIC MAPPING

Discussion

BY MESSRS. E. R. POLLEY AND LEON T. ELIEL, AND C. H. BIRDSEYE

E. R. POLLEY,[35] ASSOC. M. AM. SOC. C. E., and LEON T. ELIEL,[36] ESQ. (by letter).[36a]—This paper describes, excellently, the principles and methods involved in the making of topographical maps by the utilization of aerial photographs and the aerocartograph-measuring stereoscope. The introductory remarks of this paper indicate that Colonel Birdseye has assumed that the basic principles underlying all photogrammetric methods, have been demonstrated adequately elsewhere. Unfortunately, the writers find that these principles are not well known except by specialists. Until an engineer is able to visualize clearly how photographs are used to replace certain fundamental operations of conventional survey practice, stereoscopic methods are rather confusing at best.

This principle can best be explained from the standpoint of the familiar operations of an ordinary ground survey.

The fundamental principle upon which measuring stereoscopes operate is simply triangulation. This principle remains the same regardless of whether terrestrial photographs or aerial photographs are used, but it is demonstrated most easily by reference to terrestrial photographs.

Imagine that a survey by triangulation is contemplated of a dam site, such as that shown in Fig. 22. Many points on the north side of the canyon are to be located by triangulation from a base line on the south side. For example, assume Point X to be any point on the surface of this ground.

The ordinary procedure would be first to lay off a base line at a convenient location on the south bank, possibly 100 ft. in length. Then angles would be turned successively from the extremities of this base line to the

NOTE.—The paper by C. H. Birdseye, M. Am. Soc. C. E., was presented at the meeting of the Surveying and Mapping Division, Sacramento, Calif., April 24, 1930, and published in January, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1932, by Messrs. Theron M. Ripley, O. S. Reading, and Lowell O. Stewart; May, 1932, by Messrs. F. H. Peters, W. H. Crosson, Dwight F. Johns, and Douglas H. Nelles; August, 1932, by Messrs. R. E. Ballester and T. P. Pendleton; and September, 1932, by Otto Lemberger, Assoc. M. Am. Soc. C. E.

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[36a] Received by the Secretary October 31, 1932.

various points (of which Point X is an example), and the positions of each point would be calculated from the resultant data.

Next, assume that two transits are set up simultaneously at the extremities of this base line and that by a system of prisms the transitman stands in the middle of the base line, looking through the right transit with his right eye and the left transit with his left eye. Thus, the transits, when directed to the same object give an effect similar to that observed through a pair of

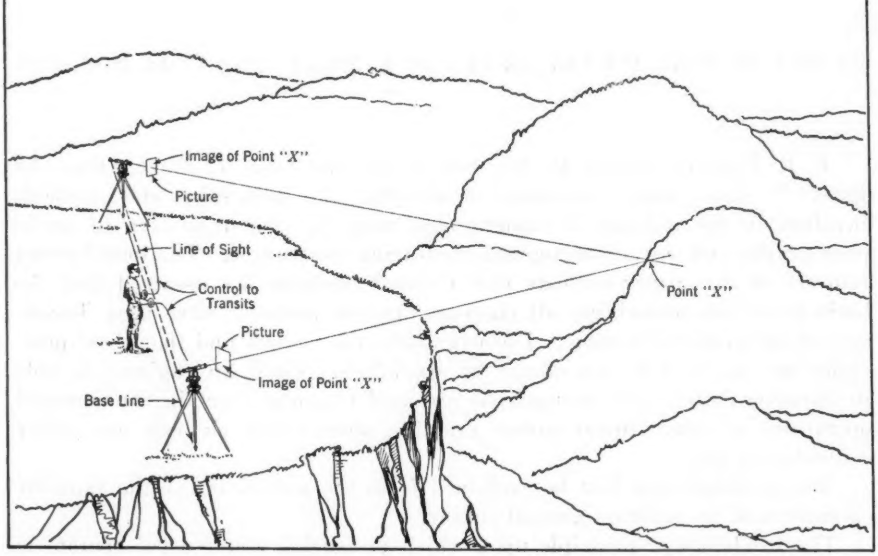

FIG. 22. PICTORIAL DEMONSTRATION OF STEREOSCOPIC PRINCIPLES.

binoculars. The observer is not conscious of looking through two separate instruments. He enjoys a marvelous capacity for relief perception, because effectively his eyes are separated 100 ft. instead of 2½ in., as is the case when observing with the unaided eye. To understand the remarkable stereoscopic effect he secures by this means, consider similar effects from every-day experience.

When looking through a pair of binoculars, human vision and depth perception (stereoscopic vision) are greatly improved for two reasons: First, the binoculars give a high degree of magnification; and, second, they make it possible to judge differences in distance with much greater accuracy. This latter phenomenon exists because the binoculars increase the base line of the eye from about 2½ in. to possibly 5 in. This practically doubles the accuracy with which relative distances can be judged. Therefore, a superlative effect can be secured, if, instead of looking through binoculars with a 5-in. base, the

observer looks through binoculars with a 100-ft. base, such as is secured with the two transits. The transitman perceives the most minute differences in distance. If one of two objects is only a few inches closer than the other, they appear to be separated definitely. This ability to judge easily slight differences in distance is effective up to distances of 1 000 to 2 000 ft., with transits separated 100 ft.

The two cross-hairs of the transits appear to merge into a single cross-hair, the center of which, lying in the lines of sight of the two instruments, appears to rest exactly at the point of intersection. In fact, the cross-hairs appear to stand in space as a vertical grid, the center intersection of which appears to rest exactly at the point of intersection. If the two transits are directed at Point X, the center of this grid (the cross-hairs) will appear to rest exactly on that point. Below this, the grid appears to have receded under the ground. Now, if the convergence of the two transits is increased slightly, so that the intersection occurs perhaps 1 ft. nearer the base line, the transitman will still see Point X stereoscopically. The only change which this increase in the convergence angle of the two transits causes, is an apparent movement of the vertical grid (the cross-hairs) so that it appears to stand nearer the transitman than Point X. If the angle of the two transits is diverged so that the point of their intersection occurs behind Point X, the grid will appear to have receded into the ground. To the observer it will appear as if the side hill were composed of transparent jelly with a grid of wire appearing first in front of the surface of the jelly and subsequently buried in it.

In similar fashion the transits can be directed at any point or object and the apparent coincidence between this object and the cross-hair grids will permit precise setting of the transits on this object.

Next, assume that the observer replaces each of the transits temporarily with a camera and takes a picture with the center of the camera lens exactly at each transit station. Then, assume that he replaces the transits and supports the pictures directly in front of them, taking extreme care to make the distance between the pivot point of the transits and the principal point of the photograph precisely the same as the focal length of the camera that took the pictures.

Now, if the observer directs the transit at the tiny dot on the picture, that is the photographic reproduction of Point X, and then removes the photograph from in front of the transit, he will find the transit pointing directly at Point X on the hillside. Therefore, the observer can direct his transits to Point X, or to any other point he wishes, by observing the corresponding image point on one photograph through one transit and the same point as shown on the other photograph through the other transit.

The application of the stereoscopic principle in the measuring stereoscope might be described very broadly by stating that the pictures are brought to the office and set up before the two observing systems of the aerocartograph instead of in front of two transits.

The foregoing description refers to photo-theodolite or ground pictures for which length of base and angles at which pictures are taken, are known.

Therefore, it is relatively simple to reproduce the relative positions and separation (all to scale) in the aerocartograph. In the case of aerial pictures, the camera is supported by a rapidly moving airplane; the base line is the distance traveled by the airplane in the interval between exposing two photographic plates. This distance is not measured; camera angles are unknown. Therefore, it is necessary to know the elevation and horizontal positions of several points on the ground photographed in order that the base-line distance and camera angles can be reproduced in the aerocartograph. There is no other essential difference; the fundamental principle is simply photographic triangulation regardless of whether aerial photographs, ground photographs, or combinations of the two, are used.

C. H. Birdseye,[37] M. Am. Soc. C. E. (by letter).[37a]—Since the presentation of this paper there have been many new developments in stereoscopic mapping instruments and methods, and some changes in the construction and use of the aerocartograph. A German book on photogrammetry,[38] has been translated into English. It is unfortunate that another book[39] has not been translated, because the text is a valuable contribution to the literature on this subject. In the United States, Earl F. Church, Assoc. M. Am. Soc. C. E., has extended his papers on topographic mapping from aerial photographs to five, all published by Syracuse University, Syracuse, N. Y. These publications have not been mentioned in the discussion of this paper and they add to the bibliography available on the subject.

As stated in the Synopsis of the paper, the writer described the aerocartograph in detail because it is the instrument with which he is most familiar. The only American-made instrument of this type was described in a previous paper[6] and, therefore, the writer made only brief mention of it. Mr. Pendleton cites the principal differences between the American and European instruments and comments effectively on the relative merits of film and plates for accurate work. The writer agrees with him in so far as large-scale work is concerned, but realizes that use of glass plates on small-scale mapping of large areas is relatively slow and expensive.

Messrs. Polley and Eliel call attention to the fact that many engineers may not be familiar with the basic principles underlying photogrammetric methods. The writer acknowledges the inadequacy of his reference to another source[3] for demonstrations of the general principles underlying the stereoscopic use of photographs, and he should have expanded other references given in the paper,[4] in the discussion by Lieutenant Reading,[11] and in this closing

[37] Acting Chf., Div. of Engraving and Printing, U. S. Geological Survey, Washington. D. C.

[37a] Received by the Secretary November 12, 1932.

[38] "Photogrammetry—Collected Lectures and Essays," Edited by O. von Gruber, translated from the German original, by G. T. McCaw and F. A. Cazalet, pub. by Chapman & Hall, London, England, 1932.

[39] Handbuch der Wissenschaftlichen und Angewandten Photographie, Band VII, "Photogrammetrie und Luftbildwesen," by Dr. R. Hugershoff, pub. by Julius Springer, Vienna, Austria, 1930.

[6] "Aeroplane Topographic Surveys," by George T. Bergen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 627.

discussion.[38, 39] However, the writer is glad that Messrs. Polley and Eliel have added such a clear and simple demonstration.

Mr. Lemberger comments briefly on the other European space autographs and presents a most interesting description of Dr. Aschenbrenner's nine-lens camera and accompanying small-scale mapping instruments and methods. The writer has not seen these instruments, but believes that the nine-lens camera idea has some advantages over the five-lens arrangement because of the filling of the gores between the wing pictures. He is doubtful, however, as to the sharpness of the images on the negative gathered by prisms into eight of the lenses. At any rate, the multiple-lens idea is certainly the solution of small-scale mapping.

The Corps of Engineers, U. S. Army, is now (1932) conducting experiments with stereoscopic plotting from five-lens photographs and, also, with the Geological Survey, is interested in tests that are being made with the new German stereo-planograph and quadruplex (four-lens) camera. The latter development has occurred since this paper was written. The camera has four magazines, all taking oblique negatives, and the stereo-planograph has been equipped with two quadruplex plate-holders so that each set of four negatives, without any transformation to the horizontal or even making glass positives, is placed in one of the plate-holders with exactly the same relations between the four negatives that were maintained in the camera. The field of each of the four negatives taken at each exposure overlaps the field of each of the others, so that there are no gores between them. The stereoscopic operator moves the pointer from one segment to another as if the composite negative was made by a single lens. The short focal length of the lenses ($13\frac{1}{2}$ cm.) permits covering a relatively large area in a single exposure. On the test of an area in Pennsylvania, a single exposure taken at an altitude of about 14 000 ft. above ground, covered an area of about 20 sq. miles on a scale of 1:30 000.

Mr. Ripley comments on the lack of a statement as to the saving of time by the use of an aerial survey. It is difficult to make a general statement because of the great difference in projects as regards scale and detail desired, and because of great differences in types of aerial surveys. The writer assumes that Mr. Ripley refers to contour maps in the two examples cited, in which case the time saved should depend largely on the scale of the map. Single-lens stereoscopic equipment heretofore used in the United States has been restricted to scales of 1:24 000, or larger. With this equipment the writer would hesitate to compete with a skilled ground topographer on a topographic survey of an easily accessible area on a scale of 1:48 000, or smaller. However, if multiple-lens equipment will permit accurate delineation of relief, the photo-topographic engineer should be able to give the ground surveyor "cards and spades" and come out ahead in a small-scale survey of an inaccessible area in which the culture is not obscured by timber cover.

[3] "Aeroplane Topographic Surveys," *Transactions*, Am. Soc. C. E., Vol. 90 (1927), pp. 627–655; also, *Bulletin* 788, U. S. Geological Survey, pp. 384–398.

[4] *Professional Paper No. 4*, British Air Survey Committee; see also, "Surveying from Air Photographs," by Capt. M. Hotine, pub. by Richard R. Smith, N. Y.

[11] "Surveying from Air Photographs," by Capt. M. Hotine.

Photographs completely covering a project may be taken in a day, but the photography itself is a relatively small part of the job. To secure the necessary ground control usually takes longer than the photographic work; but the total time consumed on an average project for which the use of aerial photographs is suitable, should be considerably less than that required to map the area solely by ground-survey methods.

Lieutenant Reading raises much the same question but specifies relative costs and technical advantages and asks for comparisons. The writer will attempt to give these from a combination of actual and estimated data.

In mapping Great Smoky Mountains National Park to a scale of 1:24 000, and with a contour interval of 20 ft., adequate ground control was available. A heavily timbered area of about 100 sq. miles was mapped by means of the aerocartograph under contract, at a price of $125 per sq. mile, resulting in a small profit. Similar areas in the same region were mapped by the U. S. Geological Survey by the plane-table method at an average cost of $148 per sq. mile for topography alone, after the fundamental ground control had been established. Had the Geological Survey used the multiple-lens camera to delineate the culture and drainage and then added the contours by means of the plane-table, it is estimated that the cost would have been much less than with the plane-table alone. With such a method gaps in trails obscured by the timber would have been added by the topographer in the field, but the difficulty the topographer had in forcing his way through the dense brush along the ridges and creeks would not be eliminated, and elevations of features off the traverse lines would still be sketched in with less exactness than resulted from aerocartograph mapping.

In mapping Zion National Park, in Utah, to a scale of 1:24 000, with a contour interval of 50 ft., adequate control was also established in advance. The area mapped was 305 sq. miles. Of this area, 157 sq. miles were covered by a topographer extremely skilled in mapping cliff areas by what is known as the station method; that is, plane-table intersections with vertical angle elevations. Climatic conditions prevented the aerial photographic contractor from delivering the photographs until near the end of the field season, but the topographer was able to identify enough control on the prints so that the remaining area of 148 sq. miles was mapped in the office on the aerocartograph. An interesting result was that the contours of the two surveys joined with practically no adjustment. The average cost per square mile of the field mapping by plane-table was $44 per sq. mile and the cost of the aerocartograph mapping was $51 per sq. mile. This area was ideal for stereoscopic mapping, but the project was somewhat experimental because the stereoscopic operators had had no previous experience in mapping cliffs. If an automatic plotting instrument had not been available, the writer would still have used single-lens photographs and executed the field plane-table work in much the same way, except that he would spend no time whatever in the field in delineating all the minute detail of slopes and cliffs. He would not prepare a base map in advance because of the delay in securing the required picture control after the photographs had been taken.

Another photo-plane-table method is that of sketching the details on the photographic prints and adjusting the data into map form later.[40] This method was used very successfully on a contract survey of Flathead Lake, Montana, on which 2-ft. contours, both under and above water, were required between extreme low and high-water limits. The preparation of a base by the radial-line method would have required waiting for field control and picture point identification work, and even with these data available the topographer would have only the shore line and the few roads, houses, and streams on his base sheet. Use of the method permitted the topographers to leave Washington as soon as the photographer wired that the area had been photographed and, by the use of air mail both ways, the topographers had a few prints available to work on the day they reached the project. The particular advantage of using this method was the elimination of almost all traversing and the substitution of the use of two short base lines on each print for scale and rectification, and the use of image points to which only vertical angles were read. The under-water forms showed perfectly in the photographs, so that the two 2-ft. under-water contours were delineated by means of a very few soundings on each print. The writer does not have at his disposal the unit cost of this survey, but he is confident that it could not have been executed as quickly or cheaply, or as well, by any other method.

The discussions by Major Johns, Captain Crosson, and Messrs. Ballester and Stuart bring up points that are answered at least briefly in the foregoing comments. Mr. Peters gives an excellent outline of what has been done in this field by means of other instruments and methods. The writer hopes that such of these instruments and methods as are now in use may be made the subject of additional papers.

From the preceding discussion it is apparent that the writer does not believe that any one method or instrument is best adapted to all kinds of mapping projects. There are so many different types of mapping projects that an organization engaged in general topographic mapping should have at its disposal as wide a range of equipment as is practicable and, above all, should have on its staff experts able to select the method best adapted to each particular project. There are few areas in the United States in which some use of photography, either aerial or terrestrial, will not produce a better map than can be obtained by ground-survey methods alone, and, in most areas, there will be a distinct saving in time and cost by using some combination of photographic and instrumental survey methods. An extremely valuable by-product of the survey will be the availability of the photographs for other uses.

[40] "Aerial Photography as an Aid in Map Making," by Gerard H. Matthes, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E. Vol. 86 (1923), pp. 779–802.

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DISCUSSIONS

DETERMINATION OF PRINCIPAL STRESSES IN BUTTRESSES AND GRAVITY DAMS

Discussion

By Messrs. Herman Schorer, A. Floris, W. C. Huntington, and Calvin V. Davis

Herman Schorer,[35] Assoc. M. Am. Soc. C. E. (by letter).[35a]—The basic equation, accepted and used by the author throughout his paper, is the ordinary column formula for eccentric loading, which, in turn, also includes the assumption of linear, vertical, normal stress distribution. In his "Conclusion," under the heading "Distribution of Stresses," Mr. Holmes states that it is well known that the vertical normal stress distribution of gravity dams is not linear, although for triangular dams of thicknesses of 50 to 60 ft., the error probably is small. When, and if, a more exact distribution of the vertical normal stress is expressed in terms of the base length, he declares it should be inserted in the formulas presented in his paper, and the shear and horizontal normal stresses will then not be distributed linearly.

These assumptions and statements are rather vague and even contradictory. The application of the ordinary column formula is scarcely adapted to the production of any new results. For the special case of a triangular dam slice of constant thickness, unlimited height, and with the water level at the crest, Levy showed,[36] more than thirty years ago, that the trapezoidal stress distribution conforms to the exact theory of elasticity. This result has since been confirmed by numerous other investigators.

The determination of principal stresses in tapering buttresses, based on the assumption of linear stress distribution in normal sections, was first

Note.—This paper by W. H. Holmes, Assoc. M. Am. Soc. C. E., was published in January, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1932, by Messrs. W. J. Stitch, Hakan D. Birke, Dirk A. Dedel, Fred A. Noetzli, and Eugene Kalman; August, 1932, by I. M. Nelidov, Assoc. M. Am. Soc. C. E.; and October, 1932, by Messrs. H. E. von Bergen and, Howard L. Cook.

[35] Chf. Engr., Thebo, Starr & Anderton, Inc., San Francisco, Calif.

[35a] Received by the Secretary October 18, 1932.

[36] "Sur l'equilibre elastique d'un barrage en maçonnerie a section triangulaire," by Maurice Levy, *Comptes Rendus* de l'Academie des Sciences, Paris, 1898.

treated[37] by Mr. B. F. Jakobsen, who also gave a full description of the application of Mohr's stress circle.

It seems rather doubtful that any further progress in the analysis of dams, of the type treated by the author, can be realized without the aid of the fundamental elastic equations. The writer has derived,[38] the general differential equation of a symmetrical buttress or slice of variable thickness, subject to gravity action and external loads, acting along the rims in the direction of the middle plane. This equation indicates that the buttress shape has a pronounced influence on the stress distribution law.

It can be shown that the assumption of a linear stress distribution will satisfy the differential equation in the case of very simple disk shapes, with correspondingly simple loading, such as the triangular gravity dam or buttress shapes of constant thickness. In these special cases, the author's results can be given in simple and general equations by means of an Airy stress function.

A. Floris,[39] Esq. (by letter).[39a]—In this paper an attempt is made to derive algebraic expressions for the stress components in buttress and gravity dams by the aid of Professor Cain's analysis. In other words, the author utilizes this numerical method for the purpose of obtaining general algebraic formulas. The question now arises, whether such a method of procedure is preferable to that of Professor Cain.

It is quite common among engineers in practice to use numerical expressions when the algebraic ones become too cumbersome. In fact, in such cases, it is the only method that leads to quick and easy answers. From this point of view, therefore, it seems that the author's efforts are in vain. His formulas are lengthy and awkward, while the corresponding numerical expressions in specific cases are simple and easy to handle. For this reason the writer doubts the usefulness of this method of approach.

W. C. Huntington,[40] M. Am. Soc. C. E. (by letter).[40a]—The writer has been interested in this paper in so far as it applies to the calculation of the principal stresses in dams of gravity section. The method presented by the author is a valuable extension of that developed by the late William Cain, M. Am. Soc. C. E., for determining the principal stresses in dams of gravity section and is not limited in its usefulness to profiles in which the up-stream face is vertical, as is the case with the elegant solution formulated by Mr. E. C. Hill.[41] In deriving an expression for the unit shear at any point in a gravity dam, Mr. Holmes might have simplified his problem by making use of one plane instead of two. Furthermore, by an extension of the method suggested[42] by Professor W. C. Unwin, the value of the unit horizontal stress at a point

[37] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 276.
[38] *Loc. cit.*, Vol. 96 (1932), p. 725.
[39] Civ. Engr., Los Angeles, Calif.
[39a] Received by the Secretary October 14, 1932.
[40] Prof. of Civ. Eng., and Head of Dept., Univ. of Illinois, Urbana, Ill.
[40a] Received by the Secretary October 19, 1932.
[41] *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, p. 134.
[42] *Engineering*, Vol. LXXIX (1905), p. 593.

on a vertical plane could have been obtained by using two planes rather than three as in the author's solution. This procedure, demonstrated herein, reduces the amount of calculation.

The cross-section of a straight gravity dam is illustrated in Fig. 28(*a*). A section with a length of unity perpendicular to the page is considered. The normal downward pressure on the horizontal section, *A B*, at a distance, *h*, below the water surface and the normal upward pressure on the horizontal section, *A′ B′*, at a distance, $h + \delta h$, below the water surface are also shown in Fig. 28(*a*). The total shear on the vertical area, *X X′*, is then, $q_x \delta h$ = Area *A′ X′ E′ C′* − Area *A X E C* − weight of prism, *A A′ X′ X*, consider-

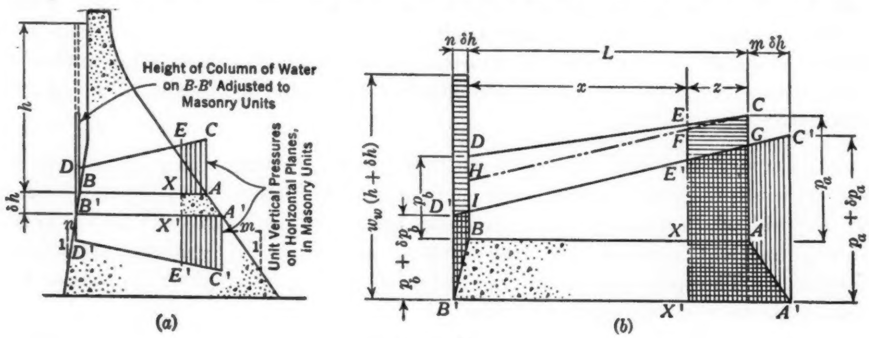

FIG. 28.

ing shear as positive, which tends to move upward the part to the right of the point under consideration with reference to the part to the left. As demonstrated in mechanics, the unit horizontal shear at a point equals the unit vertical shear at that point.

The part of the dam under consideration is shown on a larger scale in Fig. 28(*b*) in which, the pressure diagrams are plotted in masonry units and the diagram for the upward pressures on *A′ B′* is placed above that plane instead of below it, as in Fig. 28(*a*). The area of the trapezoidal block, *A A′ X′ X*, represents the weight of that block in masonry units. Since the upward and downward pressure diagrams have been superimposed, the parts of the diagrams that overlap cancel each other. From the parts that do not overlap, the shear at any point can be determined. For instance, the shear at the down-stream toe, *A*, is equal to the area, *A A′ C′ G*, and, discarding the products of differentials, is $q_a \, \delta h = p_a \, m \, \delta h$, or,

$$q_a = p_a \, m \quad \text{.......................} (132)$$

Similarly, the shear at the up-stream toe is equal to the water load on *B B′* minus the area, *B B′ D′ I*, or, $q_b \, \delta h = w_w \, h \, n \, \delta h - p_b \, n \, \delta h$. Solving,

$$q_b = (w_w \, h - p_b) \, n \quad \text{....................} (133)$$

the weight of water per unit of volume being $w_w = w \div k$.

The shear on any section, *X X′*, at a distance, *x*, from the up-stream toe, or *z* from the down-stream toe, is equal to the algebraic sum of the force on the right of that section, upward forces being considered as positive. There-

fore, referring to Fig. 28(*b*), $q_z\,\delta h =$ Area $A\,A'\,C'\,G$ − Area $E'\,G\,C\,E$. If $C\,H$ is drawn parallel to $C'\,I$, the shear is expressed by $q_z\,\delta h =$ Area $A\,A'\,C'\,G$ − Area $E'\,G\,C\,F$ − Area $F\,C\,E$, or,

$$q_z = q_a - \frac{\text{Area } E'\,G\,C\,F}{\delta h} - \frac{\text{Area } F\,C\,E}{\delta h} = q_a - r - s$$

As illustrated in Fig. 29(*a*) the area of the parallelogram, $E'\,G\,C\,F$ (designated by r), increases directly with the first power of z; hence, in the shear diagram in Fig. 29(*a*), the values of r for various values of z will fall on the straight line, $L\,N$.

The area of the triangle, $F\,C\,E$, designated by s, varies as the square of z; so it is represented by the parabola, $L\,M$, in Fig. 29(*a*) which is tangent to

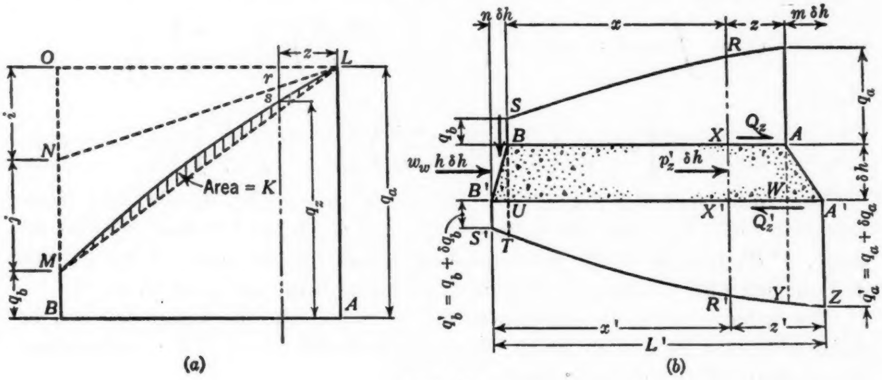

FIG. 29.

the straight line, $L\,N$, at L since the values of s are measured from $L\,N$. The value, q_z, of the shear at any point can be determined if r and s are evaluated, the line, $L\,O$, being parallel to $A\,B$.

From Fig. 29(*a*), $\dfrac{r}{i} = \dfrac{z}{L}$, or $r = i\,\dfrac{z}{L}$, and $\dfrac{s}{j} = \dfrac{z^2}{L^2}$, or $s = j\,\dfrac{z^2}{L^2}$. From geometry, the area included between the parabola, $L\,M$, and the straight line, $L\,M$, is $K = \frac{1}{2} j L - \frac{1}{3} j L = \frac{1}{6} j L$. Therefore, $j = \dfrac{6\,K}{L}$. Furthermore, the area of the shear diagram equals the total shear on the section, $A\,B$, and, therefore, must equal the total horizontal water pressure above $A\,B$. Using this relation and referring to Fig. 29(*a*), the area under the parabola is:

$$K = \tfrac{1}{2}\,w_w\,h^2 - \tfrac{1}{2}\,(q_a + q_b)\,L \quad \ldots\ldots\ldots\ldots\ldots\ldots (134)$$

The value of K may be either positive or negative depending on the dimensions of the section under consideration. From Fig. 29(*a*), $i = q_a - q_b - j = q_a - q_b - \dfrac{6\,K}{L}$, and $q_z = q_a - r - s = q_a - i\,\dfrac{z}{L} - j\,\dfrac{z^2}{L^2}$; or,

$$q_z = q_a - \left(q_a - q_b - \frac{6\,K}{L}\right)\frac{z}{L} - \frac{6\,K}{L}\,\frac{z^2}{L^2} \quad \ldots\ldots\ldots\ldots (135)$$

The quantity, K, can be eliminated from Equation (135) by substituting its value as given in Equation (134), but the form given in Equation (135) is probably preferable. The factor K, must be used with the proper sign.

The horizontal stress on the vertical plane through a point can be obtained by considering the horizontal forces acting on the trapezoidal element, $A\,X\,X'\,A'$, of Figs. 28(a) and 29(b) in a manner similar to that which was used in obtaining the shearing stress from the vertical forces acting on this element. Then, $\Sigma\, H = p'_z\, \delta h + Q_z - Q'_z = 0$, and,

$$p'_z = \frac{(Q'z \;-\; Qz)}{\delta h} \qquad \text{.......................(136)}$$

The total shearing force on the plane, $A\,B$, from A to X is, from Equation (135),

$$Q_z = \int_0^z q_z\, \delta z = q_a\, z - \left(q_a - q_b - \frac{6\,K}{L}\right)\frac{z^2}{2\,L} - \frac{6\,K}{L}\,\frac{z^3}{3\,L^2}$$

or,

$$Q_z = \left[q_a - \tfrac{1}{2}\left(q_a - q_b - \frac{6\,K}{L}\right)\frac{z}{L} - 2\,\frac{K}{L}\,\frac{z^2}{L^2}\right]\frac{z}{L}\,L \text{......(137)}$$

The total shearing force, Q'_z, on the plane, $A'\,X'$, is obtained from Equation (137) by using the values of q'_a, q'_b, K', L', and z' that refer to the plane, $A'\,B'$, instead of the corresponding values for the plane, $A\,B$, as given in Equation (137), some appropriate finite value being assigned to δh.

The value of p' at the down-stream toe and at the up-stream toe can be easily obtained from Fig. 29(b), thus, $p'_a\, \delta h =$ Area $A'\,Z\,Y\,W$. Discarding the products of two differentials:

$$p'_a = q_a\, m \text{ ....(138)}$$

Similarly, $p'_b\, \delta h = w_w\, h\, \delta h -$ Area $B'\,S'\,T\,U$, or,

$$p'_b = w_w\, h - q_b\, n \text{ ....(139)}$$

In applying Equations (135), (136), and (137) to obtain the principal stresses at a point, two horizontal planes are used, one at some finite distance above the point and another at the same distance below. Equation (137) gives the values of Q_z and Q'_z for these two planes for use in determining p' from Equation (136). Following the author's procedure, the value of q at the point is taken as the average of the values at points on the horizontal planes, just above and just below the point in question. Of course, a value of q that is exact, within the limits of the assumptions on which the analysis in this paper is based, can be found directly by passing a horizontal plane through the point and using Equation (135). This procedure is unwarranted, however, because the average value of q is sufficiently accurate. A satisfactory value for p at the point is obtained by averaging the values for p at corresponding points on the planes just above and just below the point. If the up-stream face of a dam is vertical, the calculations are simplified by taking the origin at the up-stream face as in the author's solution. The changes that must be made in Equations (135), (136), and (137) to suit the new origin are obvious.

If the horizontal planes are taken 1 ft. apart, as suggested by Mr. Holmes, it is necessary to carry all calculations out to a large number of significant figures, because the calculations for p' involve the difference of two large quantities that are nearly equal. Considerably larger intervals will give results that are sufficiently accurate for practical purposes, especially if there is no pronounced change in the profile in the region being investigated. The refinement represented by small intervals between the horizontal planes does not seem justified when the approximate nature of the assumptions, on which the entire analysis is based, is considered.

CALVIN V. DAVIS,[43] M. AM. SOC. C. E. (by letter).[43a]—This paper has directed the attention of engineers to the basic principles of stress analysis for concrete dams. The need for such fundamental thinking in dam design has been forcibly impressed upon the writer during the past few years. In a number of designs prepared by independent consulting engineers the writer found that investigations had been carried previously no further than the determination of the vertical normal pressures on horizontal planes. This procedure indicated that, theoretically, extremely thin buttresses would be safe, although actually impractical to construct. A complete analysis of the principal and shearing stresses, however, indicated that the intensity of the shearing and tensional stresses was sufficient to cause structural failure. The experience of the writer with these designs substantiates the conclusions of Mr. Holmes as to the great importance of making a complete analysis of the internal stresses in dams.

In analyzing the principal stresses in buttress dams the writer prefers to divide the buttress into elementary prisms and to determine the stresses on each prism through the solution of simple equilibrium equations.[44] This method has been applied successfully to the analysis of most of the important buttress dams constructed in recent years. Its advantages over the equations which are fully developed in Mr. Holmes' paper lie in the ease in which the effects of the irregularities of section commonly encountered in buttresses may be included in the analysis. Fundamentally, however, the methods are the same; the difference is one of procedure rather than of basic principle. The same results should be obtained by both procedures if applied to structural sections that are identically the same.

The writer is not greatly concerned, however, about the technicalities of precedure. Far more important than these are the improved types that are now being developed as a result of a better understanding of the stress distribution in dams. Such important and far-reaching improvements of type as the round-head buttress dam,[45] the buttress dam of uniform strength,[46] as well as the massive buttress dam previously described[47] by the writer, resulted

[43] Chf. Designer, Ambursen Dam Co., New York, N. Y.

[43a] Received by the Secretary November 13, 1932.

[44] "Safeguarding the Lake Pleasant Dam," Fred A. Noetzli, M. Am. Soc. C. E., *Western Construction News*, April 25, 1929.

[45] "The Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E., Eighth Edition.

[46] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 666.

[47] "Recent Advances in Buttress Type Dams," *Civil Engineering*, February, 1931.

directly from fundamental studies of the distribution of principal stresses in the several structural members of a dam.

Another example of structural improvement that resulted from a thorough analysis of internal stresses is afforded by a dam, near Prescott, Ariz. Maximum economy in concrete was attained in this dam by proportioning the structure in such a way that both the intensities of shearing stress and first principal stress were substantially uniform on any horizontal plane. The diagrams shown by Fig. 30 indicate the variation of the first principal stress

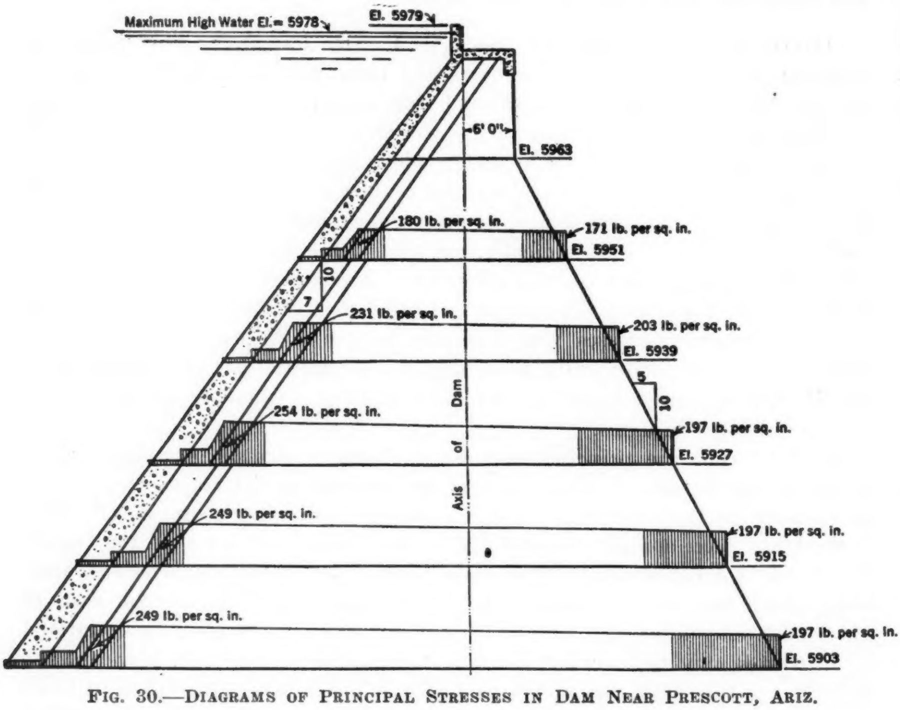

FIG. 30.—DIAGRAMS OF PRINCIPAL STRESSES IN DAM NEAR PRESCOTT, ARIZ.

in this dam. Further economy was attained through keeping the second principal stress in compression, thus eliminating the necessity for excessive inclined steel. The writer found that the uniformity of these stress functions was controllable almost entirely through the proper selection of the up-stream and down-stream buttress slopes. This important relation of buttress slopes to principal and shearing stress distribution would not have been revealed if the analysis had been carried no further than the determination of the vertical normal stresses on horizontal planes.

It may be seen from this example that both increased safety and economy may result from the practical application of any of the methods of determining principal stresses. If Mr. Holmes' paper results in a more thorough understanding of the internal stress distribution in dams, a widespread use of the improved structural types is certain to follow.

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DISCUSSIONS

DESIGN CHARACTERISTICS OF THE READING OVERBUILD TRANSMISSION LINE

Discussion

By Frederick W. Deck, Assoc. M. Am. Soc. C. E.

Frederick W. Deck,[7] Assoc. M. Am. Soc. C. E. (by letter)[7a]—In his discussion of the paper, Mr. Tratman has mentioned the objection which has frequently been made by railroad companies to overbuild propositions, concerning the chance of a derailment of trains. Accidents of this kind, as he states, are relatively of such infrequent occurrence that generally this objection must be given little weight. In the case of the power lines of the Reading Overbuild there is a further safeguard to the structure in the distance which the posts are removed from the nearest track. On a tangent section about $1\frac{1}{2}$ miles in length, the bridges that span the tracks have their posts, under present conditions, approximately 23 ft. from the center line of the nearest track. If the Railroad Company should ever find it necessary to build the third and fourth tracks along this section, the nearest track would then be only 10 ft. from the posts. That, however, is probably a remote possibility. On many of the curves where the bases of the structures are much closer to the tracks than on tangents, trains are operated at less speed and possibly with somewhat greater care.

From the point of view of the Railroad Company, there is not much more hazard in this overbuild transmission line than there would be with the slender post bents which otherwise would support the transmission line for railroad-electrification purposes.

In order to provide maximum security, it was found desirable, in this case, to make the structures and the other elements of the transmission line as strong as was reasonably necessary to stand up under the most adverse conditions of loading likely to be encountered. The design of the structures was conservative, involving comparatively low unit stresses with heavy loading assumptions. Double strings of insulators were used in strain positions,

Note.—The paper by Frederick W. Deck, Assoc. M. Am. Soc. C. E., was published in February, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1932, by Messrs. E. E. R. Tratman, and Percival S. Baker.

[7] Structural Engr., Mech. Div., Philadelphia Elec. Co., Philadelphia, Pa.

[7a] Received by the Secretary October 14, 1932.

thus adding another safety factor; and the structures themselves were designed so as to stand up under the complete dead-end pull with all the transmission wires broken.

Naturally the unity of interest of the Railroad Company and the Electric Company in the provisions for railroad-electrification facilities as well as for power-line facilities, made the arrival at an agreement somewhat easier to accomplish. However, in this particular case, the two companies were especially fortunate in being able to view the matter from a relatively clear and impartial standpoint, unencumbered by hidebound adherence to fictitious principles of safety. Every effort was made to render this transmission line safe from the standpoint of either the Railroad Company or the Power Company, but useless and obsolete design restrictions were not thrown in the path of the negotiations leading up to mutual agreement.

For more or less obvious reasons, it was not possible to quote the terms of the agreement between the Electric Company and the Railroad Company. It may be stated, however, that the Railroad Company was amply protected through a liability clause in case of accident due to the presence of electric facilities.

Mr. Baker has mentioned the work of eliminating grade crossings that was done by the Railroad Company at about the same time that the transmission line was constructed. The Electric Company was particularly fortunate in being able to co-ordinate the design and construction of the transmission line with the design and construction of the elevated viaduct of the railroad. This simultaneous process of design and construction by the two companies was a means of saving considerable time, labor, and expense, that otherwise would have been necessary had the transmission-line construction been delayed until after the railroad work was completed.

The comment may be made by some engineers, that these structures appear to be unusually heavy for the type of loads they are called upon to sustain. Upon a casual inspection of the line this certainly appears to be true, but a closer study will reveal, inevitably, the fact that under complete loading the structures are subjected to such a variety of heavy and unusual pulls, far removed from their natural points of support, that the steelwork will no longer seem too heavy. The conditions under which the work was accomplished, the limitations of the right of way, and the prerequisites of safe clearances dictated in large measure the unusual shapes that were adopted; and, consequently, large quantities of steel and concrete were required.

The Electric Company, realizing the necessity for providing structures of maximum safe design, both for its own sake and for that of the Railroad Company, did not hesitate to incur any reasonable expense for steel and concrete for arriving at this end. This attitude was rewarded by furtherance of the good relationship existing between the two companies. The spirit of co-operation and co-ordination of the work on the part of the two utilities involved in this project is worthy of note. It stands as an example of what may be accomplished and what benefits all parties may receive by the attainment of a common plane of thought and action.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

SUSPENSION BRIDGES UNDER THE ACTION OF LATERAL FORCES

Discussion

BY MESSRS. GLENN B. WOODRUFF, AND LEON S. MOISSEIFF AND FREDERICK LIENHARD

GLENN B. WOODRUFF,[21] M. AM. SOC. C. E. (by letter).[21a]—Ever since Mr. Moisseiff, in designing the Manhattan Bridge, developed the mathematics of the deflection theory, engineers have been able to calculate accurately the stresses due to vertical loads in a suspension bridge. It is gratifying that Mr. Moisseiff, in collaboration with Mr. Lienhard, has furnished an equally accurate method of determining the stresses due to horizontal loads.

The mathematical processes are developed so clearly that they are subject to no criticism. In addition, the results have been confirmed by model tests. The writer will attempt only to emphasize some of the authors' conclusions.

In any bridge, except the suspension type, horizontal displacements of the structure may cause additive stresses of a serious nature. With a suspension bridge, the same degree of rigidity is not required as with other types. Furthermore, as the authors point out, the action of the cables adds greatly to the rigidity of the system.

Additional examples may further illustrate the point. The adopted design for the San Francisco-Oakland Bay Bridge has center spans of 2 310 ft. from center to center of towers, the stiffening truss span being 2 294 ft. The horizontal moment of inertia is 750 000 in.^{2} ft.^{2}

The lateral force on the unloaded structure has been assumed as 900 lb. per lin. ft. of the suspended structure plus 135 lb. per lin. ft. of cable. The distribution of the horizontal force between the trusses and the cables, by

NOTE.—The paper by Leon S. Moisseiff, M. Am. Soc. C. E., and Frederick Lienhard, Esq., was published in March, 1932, *Proceedings*. Discussion of this paper has appeared in *Proceedings* as follows: May, 1932, by Messrs. P. L. Pratley, and Charles A. Ellis; and September, 1932, by Messrs. E. L. Pavlo, Ralph A. Tudor, F. H. Constant, Charles Derleth, Jr., A. A. Eremin, Elmer C. Osgood, and Charles M. Spofford and John B. Wilbur.

[21] Engr. of Design, San Francisco-Oakland Bay Bridge Comm., San Francisco, Calif.

[21a] Received by the Secretary October 10, 1932.

both theories, is shown by Fig. 24. The stresses by the two theories are, as follows:

	Uniform	Elastic
Moment at center of truss, in kip-feet....	329 000	340 000
Reaction at tower top, in kips............	614	451
Reaction at roadway, in kips............	573	736

The horizontal deflections of the system under different assumptions are, as follows:

	Deflections, in Feet Cables	Truss
As designed	6.7	7.5
Assuming no relief from truss ($I_h = 0$)...	13.0	14.4
Assuming no relief from cables...........	....	14.0

In one of the preliminary designs on this project a 3 800-ft. span was considered. It was desirable to use a truss spacing of not more than 72 ft.

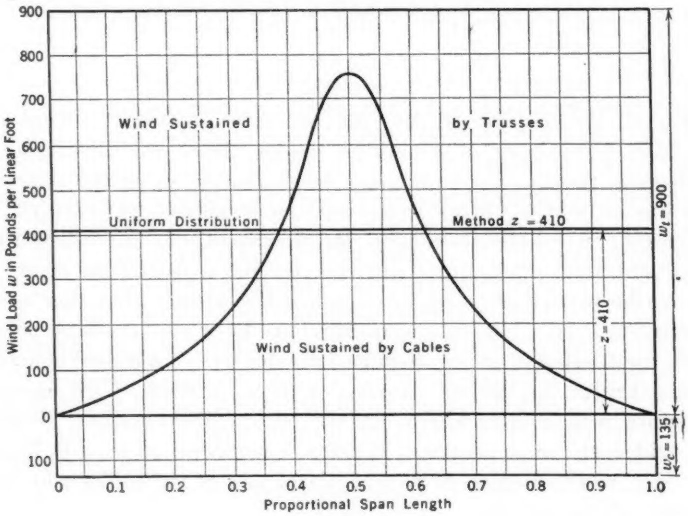

FIG. 24.

With a wind load of 1 050 lb. per lin. ft., the horizonal deflection was only 15.5 ft., or 1/245 of the span.

It is believed that the foregoing with the other examples in the paper and discussions, is sufficient to demonstrate that the usual empirical rules for truss spacing have no application to suspension bridges. It is evident that the minimum truss spacing should vary as an inverse power of the span length, and quite possibly a spacing of 60 ft. would prove sufficient for the maximum possible span.

If there is any criticism to be made of the paper it might be that the authors do not recommend more strongly that the elastic distribution method be used for all final designs, the uniform distribution method being reserved

for preliminary calculations only. As the authors point out, the difference in the moments calculated by the two methods is not great for spans of less than 1 500 ft. In the design of the lateral system, however, the uniform distribution method gives results erring on the unsafe side (see Table 3) by 13% for the Delaware River Bridge, 20% for the Detroit Bridge, and even more for the longer spans.

LEON S. MOISSEIFF,[22] M. AM. SOC. C. E., AND FREDERICK LIENHARD,[23] ESQ. (by letter).[23a]—The object in presenting this paper was to bring to the attention of engineers engaged in building long-span bridges the great lateral stability of suspension bridges, and to lay before them ready and tried methods by which they could compute the moments, shears, and deflections produced by lateral forces. The work presented is an outcome of the evolution of bridge building. With the growing demand for bridges of longer and longer spans the required width of bridge for a given long span became of decisive importance. The standard rules generally adopted by engineers for short and rigid bridges evidently do not apply to long spans; a ratio of one-twentieth of a span of 4 000 ft., according to these rules, demands a width of bridge manifestly uneconomical and impracticable. It is one of the curiosities of engineering that a few years ago a system of stiffening suspension bridges laterally by means of horizontal cables extending far beyond the trusses was proposed and patented. The matter of the satisfactory width of long-span suspension bridges extends beyond the purely engineering sphere. Some time ago one of the writers had to defend before a Court of Justice the feasibility of building a suspension bridge with a width of less than one-twentieth of the span.

As long as bridges were built with spans of moderate lengths their lateral stability and behavior was of minor importance, because the requirements for carrying capacity usually demanded widths also sufficient for structural purposes. Longer spans in some cases, such as the Firth of Forth Bridge, were given special study and treatment. Engineers who deal with longer suspension bridges know from observation that these bridges develop much lateral stability under heavy winds. This knowledge however, was one of quality rather than of quantity. To establish this quality on a scientific basis a rational theory was required so that numerical quantities could be computed. Further, to verify the predicted behavior of such bridges, observations would have to be made on bridges under the action of lateral loads. As this is nearly impossible or, at any rate, impracticable, recourse must be made to observations on suitable models of bridges. This has been done by the engineers of the San Francisco-Oakland Bay Bridge.

At the time of the design of the Manhattan Bridge, with a width of 96 ft., the strengthening of the Williamsburg Bridge, with a width of 67 ft., and the design of the Delaware River Bridge, with a width of 89 ft., it was felt that these bridges were sufficiently wide to allow for the application of

[22] Cons. Engr., New York, N. Y.

[23] Asst. Engr., The Port of New York Authority, New York, N. Y.

[23a] Received by the Secretary November 26, 1932.

a simple and direct formula for the relief extended by the cables to the trusses. The formula used was that of the Uniform Distribution Method as given in the paper. In proportioning the chords of the stiffening trusses of the Delaware River Bridge it became apparent that the wind stresses began to aspire to the control of the sectional area of the chords, and that a more profound and correct analysis of the wind effect was demanded than was offered by the uniform distribution method.

It should be explained here that in the relatively shorter spans of these bridges, chord sections of the stiffening trusses were determined by the effect of the live load and temperature, and that the wind stresses were not of sufficient magnitude to exceed the usual allowance made for wind by increasing the unit stresses. When, later, a study was made of the wind stresses on the George Washington Bridge, with a ratio of width to span of 33, and the Ambassador Bridge, at Detroit, Mich., with a ratio of 31, a more penetrating analysis was evolved, as given in the Elastic Distribution Method. The essence of this method is nothing more than the recognition of the phenomenon of the great resistance of the strained cables to distortion, on which phenomenon the deflection theory for vertical loads is based. From the standpoint of time, the uniform distribution method was not a simplification of the elastic distribution method, nor an intended approximation of it. The latter is a later and more thorough analysis of the behavior of a suspension bridge under the action of lateral forces. The elastic distribution method, as in the case of all progress of human knowledge, is a product of the necessities and demands of life. Bridges of longer span called for more accurate methods of analysis, structurally and economically, and these were brought forward in due time.

The manner of approach selected was that which appeared to the writers to be the most workable and most lucid. The choiee of mathematical tools in many instances, is a purely subjective matter depending on the technical education and mental preference of the investigator. Many roads lead to Rome, and various mathematical approaches may bring the same results. It is important that Rome be reached. The numerical results of the investigation displayed characteristics of the behavior of suspension bridges, which were not known and not expected, but which with a clearer understanding should have been foreseen. The writers refer here to the characteristic projection of the wind-distribution curve in longer spans, showing a pull at the center of the truss in the opposite direction to the wind pressure.

The use of the moment-area principle appears to be a simple and pictorial way to present the bending moments in the trusses and to determine their magnitudes. The method is absolutely correct within its assumptions and can be applied to secure results as accurate as desired. Engineers are not likely to forget, however, that they deal with wind forces the magnitude, continuity, and extent of which are uncertain, and that the specified forces are likely to vary within wide limits. To apply hair-splitting accuracy here, would be quixotic.

Mr. Ellis has developed a process of applying the elastic distribution method in which all steps are algebraic and which leads to the solution of a

number of simultaneous equations. He has established the equations for uniform as well as for partial loads; and he has also included the effect of the transverse deflection of the tower tops in these equations. The procedure is clear in its course and elegant in its construction. The "rule of coefficients" to which Mr. Ellis calls attention in establishing these equations will serve as an aid and a check of the work. By solving the simultaneous equations established for the given case of loading the "cut and try" procedure will be avoided. As stated by Mr. Ellis, both the algebraic and the writers' solutions give identical results.

In his study of the problem, Dr. Pavlo (as in the case of Mr. Ellis) has arrived at simultaneous equations. The discussions of Messrs. Ellis and Pavlo complement each other. The diagrams of distribution for partial wind loads on the George Washington Bridge are instructive. Dr. Pavlo also proposes an approximate method which is based on a convex parabolic restitution curve of the wind load, which approaches the forms of the curves shown in the paper. The proposed formula should give good results and can be readily applied.

Mr. Eremin's proposed approximation is an attempt in the wrong direction. His distribution curve is concave instead of convex as that of Dr. Pavlo and as are the plotted results of computations and tests. Mr. Eremin is mistaken in stating that "in a long-span suspension bridge the ends of the stiffening trusses are restrained, due to the action of their own weight." They are not restrained, unless especially designed in that way.

Mr. Pratley has made use of the assumption of a triangular distribution of the wind load with the peak at the center of the span. A formula based on this assumption can readily be deduced. It bears a marked resemblance to Equation (51) deduced by Dr. Pavlo for the convex parabolic assumption. The arithmetical application of either formula is equally simple, but Equation (51) will give the closer results. Mr. Pratley's question as to whether the floor system, lateral system, and truss web members are calculated as contributing to the lateral moment of inertia, in addition to the four chords, is answered in the negative. The effect of the lateral moment of inertia, of course, is not in linear proportion to the load transferred from the trusses to the cables and is not in linear proportion to the resulting moments and deflections. Its effect is rather small. In fact, the small effect of the width of the bridge is to some extent the basis of the paper. This is drastically illustrated by doubling the value of I in Example 1. The amount of wind load transferred from the trusses to the cables will change from 880 lb. to 702 lb. per lin. ft., a reduction of not more than 20 per cent.

Mr. Osgood also has chosen as his thesis the effect of lateral forces on suspension bridges. As others have done, he has sought a general analytical solution of the problem and has found it difficult of application. His findings of the relative value of the effect of the limiting assumptions made in the elastic distribution method are of interest and value.

The development and application by Professor Spofford and Mr. Wilbur of Pigeaud's paper on the action of wind on the trusses of a suspension bridge is another approach to the solution of the problem. It is scarcely the shortest.

Long-span suspension bridges are not often built, and each case is of sufficient importance to demand the full attention of the designing engineers. Tables and curves prepared beforehand may all have their use but, to become thoroughly familiar with them, the engineer must dig his way through by himself. The discussion by Professor Spofford and Mr. Wilbur is, however, an important contribution to the subject.

Professor Constant's discussion has added the valuable diagrams which show the wind distribution and the comparative chord stresses for the Mt. Hope Bridge.

Mr. Woodruff's discussion gives the interesting comparative results of wind action as computed by the uniform and the elastic distribution methods for the newest of the long-span suspension bridges, the twin bridges over the West Bay at San Francisco, Calif.

Models were built in connection with the San Francisco-Oakland Bay Bridge at the University of California to a scale of 1:100. In addition to the effect of vertical loads, the effect of lateral loads was observed carefully. It is highly gratifying to find in Mr. Tudor's discussion an experimental verification both of the phenomenon of stability and the writer's method of its evaluation.

Dean Derleth clearly brings out the structural and economical value of the lateral stability of suspension bridges, which the paper attempts to demonstrate. He shows that the application of the theory set forth, has made it possible to limit the width of the 4 200-ft. span of the Golden Gate Bridge to 90 ft., and that of the San Francisco-Oakland Bay Twin Bridges of 2 310-ft. spans to 66 ft. The economies realized by the application of the writers' theory are considerable.

The discussion which the paper has received has shown the timeliness of the subject. The writers feel indebted to the participants in this discussion for the interest taken in the paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

FORESTS AND STREAM FLOW

Discussion

By Messrs. J. C. Stevens, Harry F. Blaney, Daniel W. Mead, Ralph R. Randell, H. K. Barrows, Donald M. Baker, Ralph A. Smead, and George H. Cecil.

J. C. Stevens,[17] M. Am. Soc. C. E. (by letter).[17a]—After many years of guessing and useless argument some authentic information has been produced at last as to the effect of forests on stream flow. The final conclusions are exceedingly simple. They might have been—in fact, actually were—anticipated many years ago. The forest, like every other vegetable crop, consumes large quantities of water for its growth. Unlike small plants, however, it also dissipates large quantities of water by mechanical means. It prevents a substantial portion of rain and snow from reaching the soil, permitting rapid evaporation from branches and leaves. When the forest is removed, the water thus consumed by it appears as run-off. As new growths appear the run-off gradually diminishes again in proportion to this crop and mechanical consumption. Two widely separate tracts cited by the authors under diverse climatic conditions attest emphatically to this fact. All other effects, such as advance or delay of maximums and minimums, are quite trivial, uncertain, and inconsequential.

Writers on this subject have been to exquisite pains to avoid the acknowledgment of the foregoing simple conclusion. Why all the feverish urge to prove that forests are intimately connected with an improvement of the regimen of streams? Why must simple facts be submerged under an avalanche of conjecture when they fail to meet preconceived specifications?

The answers are perhaps best stated by Dr. John Ise[18] in recounting the history of the Weeks Bill which was introduced in Congress in July, 1909, and passed in February, 1911. About $9 000 000 was appropriated for the

Note.—The paper by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxwell, Assoc. M. Am. Soc. C. E., was presented at the Annual Convention, Yellowstone National Park, Wyoming, July 6, 1932, and was published in August, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1932, by C. G. Bates, Esq.; and November, 1932, by Messrs. J. E. Willoughby, and A. L. Sonderegger.

[17] Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

[17a] Received by the Secretary September 23, 1932.

[18] "The United States Forest Policy," by John Ise, Yale Univ. Press.

purchase of forest lands in the Appalachian and White Mountains "for the protection of the water-sheds of navigable streams."

The debate over the Weeks Bill consumed one and one-half years of Congressional and National attention. It marked an epoch in the history of the forest policy in the United States. Dr. Ise declares that the main purpose of this Act in emphasizing the assumed relation of forest conservation on the navigability of streams, was merely to meet the question of constitutionality. He points out that there is no agreement among authorities as to the effect of forests in equalizing water flow; that the benefits have been greatly exaggerated by many writers; and that such benefits were grossly exaggerated in the debates in Congress.

The following is an example of the gross exaggeration prevalent during the heyday of militant conservation:[19] "The connection between forests and rivers is like that between father and son. No forests, no rivers."

Another calamitous prophesy was to the effect that "when our forests are gone the streams will dry up, the rivers will cease to run, the rain will fall no more, and America will be a desert!"[20]

In 1910, the writer was employed on the famous "Smoke Case" by the Anaconda Copper Mining Company to make a study of the effect of forests on stream flow, climate, and soil, with particular reference to conditions in the Deer Lodge National Forest surrounding the huge Washoe Smelter, at Anaconda, Mont. This study was prompted by a suit instituted by the U. S. Forest Service to enjoin the Company from further operation of the smelter on the grounds that the smoke was destroying the surrounding forest and that the destruction of the forests would result in damages amounting to untold millions to the valley of Clark Fork River in increasing its floods, diminishing its minimums, and filling its valley with the soil from countless acres of denuded forest areas. The picture set up in the complaint was very gloomy.

The writer entered this study with an open mind, if anything, somewhat biased in favor of the forest-water-supply theory by reason of having absorbed most of the propaganda extant at the time. He gathered all the long-time stream-flow records in the world on which decided changes in forest cover have taken place. Among them were the Tennessee, Ohio, Ottawa, Murray (Australia), Merrimac, Sudbury, and Croton Rivers. No changes in regimen that by the greatest stretch of the imagination could be attributed to changes in the forest cover, could be detected. Any cyclic variations disclosed in run-off was accounted for readily and simply by similar variations in climatic factors. The generalized results of this study have been published.[21] After this study the writer felt fully convinced that the forest-water-supply theory belongs in the catalog of political expediencies rather than that of the sciences. Man's efforts to change the regimen of streams are puny and

[19] "The Fight for Conservation," by Gifford Pinchot, Doubleday, Page & Co., New York, 1910.

[20] *Scrap Book*, September, 1908.

[21] "Forests and Their Effect on Climate, Water Supply, and Soil," by J. C. Stevens, M. Am. Soc. C. E., *Journal*, Assoc. of Eng. Societies, July, 1913.

futile. The governing factors belong to forces far beyond his power to control.

Recently, the writer had occasion to go over a part of the lands around the Washoe Smelter, that were the subject of such intense study twenty-two years ago. The smoke has killed practically all the trees leeward of the smelter for many miles. Of course, Government and private owners have been amply compensated for their loss. The streams flowing from this graveyard of trees still run crystal clear and sparkling. There is no diminution in the supply as far as ocular evidence can be trusted. There is no more evidence of soil erosion than existed while the trees were alive and green.

Every one loves the forests; every one is glad they are in the hands of the Government and is grateful that they are being conserved for this and future generations. Their value for timber supplies, for recreational playgrounds, and to soothe æsthetic senses are ample and cogent reasons for preserving that control. If it was necessary once to attribute to them properties they do not possess, in order to "kid" the people and Congress into doing the thing that everybody wanted done anyway, that necessity has passed. Forest control is now a blessed fact. Is there any further necessity to keep up the farce?

HARRY F. BLANEY,[22] ASSOC. M. SOC. C. E. (by letter).[22a]—Although European and American experimenters have been observing the influence of forests on stream flow for many years, no final conclusions covering the entire subject have been definitely established.

One needs only to review some of the voluminous literature on this subject to be convinced that there is a great difference of opinion among authorities. For instance, some believe that because rainfall is most abundant where forests grow, that they exert an important influence on the amount of precipitation, and that it is a definitely established fact that the destruction of forests is followed by a decrease in rainfall and drying up of streams and springs. Others have reached the conclusion[23] that "precipitation controls forestation but forestation has little or no effect upon precipitation. * * * The run-off of our rivers is not materially affected by any other factor than precipitation."

Thus, any new data that may throw additional light on this perplexing subject, are welcomed by the engineer, forester, and agriculturist. The authors are to be complimented on their excellent paper. The writer will confine his discussion to the part pertaining to Southern California, as most of his experience has been in that section.

Some investigators have been forced to the conclusion that "in respect to run-off, each stream is a law unto itself." This, undoubtedly, is true in many instances. There are so many complex factors influencing stream flow that the problem of determining the effect of forests on run-off by comparison

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[22] Irrig. Engr., Bureau of Agricultural Eng., U. S. Dept. of Agriculture, Los Angeles, Calif.

[22a] Received by the Secretary October 4, 1932

[23] A Report on "The Influence of Forests on Climate and on Floods," by Willis L. Moore, House of Representatives, U. S. Committee of Agriculture, 1910, p. 37.

of the discharge of streams of forested and denuded water-sheds is extremely difficult. Run-off is influenced so largely by the character of the precipitation, topography, elevation, geology, vegetation, and artificial control that quantitative measurements of the effect of forests on the flow of streams by comparing forested and deforested areas are usually open to criticism. However, the authors have adopted ingenious methods to overcome most of these objections.

A discussion of "forests and stream flow" would not be complete without including the subject of consumptive use of water by native vegetation. That a gap exists between the quantity of rain falling on a given water-shed and the total run-off is clearly indicated by an analysis of data shown in Tables 2 and 3. For the period under consideration, the minimum annual run-off of the Santa Anita Creek occurred in 1924-25 and was 1.24 in., or about 6% of the total precipitation. In other words, the precipitation unaccounted for in 1924-25 was 20.67 in., or 94%; while in 1921-22, it was 30.62 in., or 51 per cent.

Whether this difference was due entirely to evaporation and transpiration losses is a debatable question. The authors state that it is doubtful whether any under-flow passes either station. Run-off, according to Meyer,[24] constitutes the residual precipitation after evaporation, transpiration, and deep seepage losses have been supplied, and the demands of evaporation and transpiration require from about 15 to 25 in. of precipitation per annum. Zon states[25] that the water consumed by the forest is nearly equal to the total annual precipitation—in cold and humid regions less, and in warm and dry regions somwhat greater.

In evaluating the evapo-transpiration losses of drainage basins, such as Santa Anita Creek and Fish Creek, it is desirable to divide the water-shed into two areas: (1) Drained slopes; and (2) canyon bottom. The "drained slopes" or sides of the canyon usually support chaparral, while in the "canyon bottom" water-loving trees, such as alders, willows, and sycamores, predominate. Evapo-transpiration losses from "drained slopes" start with the first rains in the fall and continue throughout the rainy season into the summer, until all the moisture in the soil, available for plant growth, has been extracted. It is difficult to determine these losses by field measurements. However, the consumptive use of water by brush cover on the valley floors has been successfully measured by soil-sampling methods.[26] Evapo-transpiration losses in the "canyon-bottom" area continue throughout the summer months, or as long as water is supplied from deep springs or is available from storage in alluvial fill. Under favorable conditions these losses can be measured.

The conservation of water by removing brush from the "drained slopes" is remote; the resulting damage by erosion would more than offset the value

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[24] "Elements of Hydrology," by Adolph F. Meyer, M. Am. Soc. C. E.

[25] "Forests and Water in the Light of Scientific Investigation," by Raphael Zon, Forest Service, U. S. Dept. of Agriculture, 1927, p. 20.

[26] "Rainfall Penetration and Consumptive Use of Water in the Santa Ana River Valley and Coastal Plain," by Harry F. Blaney, Assoc. M. Soc. C. E., C. A. Taylor, and A. A. Young, Assoc. M. Am. Soc. C. E., *California State Bulletin No. 33.*

of the water saved. However, conservation by reducing the evaporation and transpiration losses in "canyon-bottom" areas of a water-shed is more promising. The City of San Bernardino, Calif., is accomplishing this by piping water out of the stream channel. In Southern California this expense is justifiable since title to water diverted by gravity from streams has a market value of from \$2 000 to \$4 000 per miner's in. (continuous flow), or \$100 000 to \$200 000 per sec-ft., depending on its use.

Little information is available as to the quantity of water lost in the bottom of canyons through evaporation and by transpiration from vegetation. In the spring of 1929 some experiments were conducted by the U. S. Bureau of Agricultural Engineering on the "consumptive use of water by native vegetation along stream channels" [26] in Temescal Canyon, 4 miles southeast of Corona, Calif. The vegetation consisted of willows, tules, and kindred moist land growths. For a 30-day period, from April 28 to May 27, inclusive, the total evapo-transpiration was 12.9 acre-in. per acre.

In 1931, this investigation was extended to Coldwater Canyon[27] near Arrowhead Springs, Calif., and methods were developed to measure the evaporation and transpiration losses in canyon bottoms.

One of the methods used was to measure losses of stream flow between two bed-rock controls, located about 2 100 ft. apart in the canyon. The approximate elevations of these controls above sea level are 2 300 and 2 500 ft., respectively. The canyon between the controls is narrow and has precipitous sides, with the bottom width ranging from 30 to 50 ft., and the vegetation is mostly alders, with some sycamores, bay, oak, and herbaceous growth. During the period of record, there were no visible indications of water reaching the section under study other than that passing the upper control. At each control a low concrete dam was built across the stream channel on a bed-rock foundation. A 3-in. Parshall measuring flume, equipped with a Stevens type L water-level recorder, was placed at one end of each dam for the purpose of measuring the flow of water passing into and out of the experimental section of the canyon between the controls. At first, values for the gauge height for each hour were used in computing the volume of water passing the two controls. The daily loss between controls was obtained by subtracting the daily volume of water passing the lower control from the daily volume passing the upper control. This proved to be a lengthy and laborious process, and in order to eliminate a large part of this routine work, a flow recorder was attached to the water-stage recorder. This flow-recorder attachment consists essentially of a spiral cam that solves the flow formula mechanically.

The loss of water per day between the upper and lower controls may be obtained directly by superimposing the two flow-recorder charts, one from each control, and measuring the area between the two curves by planimeter. This area represents the daily loss between the controls, and when multiplied by the proper constant can be converted into the hydraulic units desired.

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[27] An unpublished Progress Report on Evaporation and Transpiration Studies in Coldwater Canyon, by Harry F. Blaney, Assoc. M. Am. Soc. C. E., C. A. Taylor, and H. G. Nickle, Jun. Am. Soc. C. E., Bureau of Agricultural Eng., U. S. Dept. of Agriculture, 1932.

The relative amounts of the indicated losses that are chargeable separately to evaporation and transpiration are difficult to determine. However, Rohwer[28] has indicated that the rate of evaporation is relatively low when the temperature of the water is below that of the overlying air. This condition exists in Coldwater Canyon throughout the growing season as the water comes from cool springs and is shaded as it flows through the stream channel of the canyon. Thus, evaporation is small, and the loss indicated by the daily drop in stream flow is due principally to transpiration.

From the Coldwater Canyon measurements it is estimated[27] that the consumptive use of water by canyon-bottom vegetation during the 6-months summer season, May to October, was 45 acre-in. per acre, 238 acre-in. per mile of canyon, or a depth of 0.10 in. over the water-shed per mile of canyon.

Referring to data given by Messrs. Hoyt and Troxell, the average run-off from Santa Anita Canyon during the six summer months, May to October, inclusive, for seven years ending September, 1924, was 1.19 in. in depth over the entire water-shed. The stream in the Santa Anita Canyon (including the tributaries) is estimated to be 13 miles long. Assuming that the data obtained from the Coldwater Canyon experiment can be applied to Santa Anita Canyon, the use of water by canyon-bottom vegetation would be 1.30 in. in the latter. This amounts to 109% of the average run-off for the same period or, in other words, the run-off for the six summer months would be increased 109% if no water was used by the canyon-bottom vegetation.

In like manner, an estimate can be made of the summer consumptive use of water in Fish Canyon. From the Fish Creek water-shed for the 6-months' period—May to October—for seven years ending September, 1924, the average run-off in depth, in inches, was 0.90. The stream is estimated to be 10 miles in length, and the use of water by canyon-bottom vegetation, 1.0 in., or 111% of the average run-off for the 6-months' summer period for seven years preceding the fire. This result is somewhat surprising since the authors show an annual run-off of only 1.00 in. in Fish Canyon the year before the fire, and an annual normal run-off of 1.07 in. the year following the fire. In other words, for these years the summer use of water by canyon-bottom vegetation was 100% of the total annual run-off. The increased run-off for the six months during the summer following the fire was only 0.40 in. Thus, if all the water used by canyon-bottom vegetation during the summer months could be conserved by piping it out of the stream channel, the increased run-off would be 1.0 in., or 250% of the increased run-off resulting from the burning off of the water-shed. However, it is doubtful whether more than 80% of the water consumed by the canyon-bottom vegetation could be saved.

In closing, the writer would like to call attention to the serious problem of erosion in Southern California. The data given in Table 12 of the paper illustrates clearly the large quantity of silt carried by a stream after the vegetation has been burned off a small water-shed. It should be pointed out that the rainfall that year was about 30% below normal; and, thus, only a

[28] "Evaporation from Free Water Surfaces," by Carl Rohwer Assoc. M. Am. Soc. C. E., *Technical Bulletin No. 271*, U. S. Dept. of Agriculture, December, 1931.

minimum of damage was done. However, in a year like 1921-22, with an annual precipitation of 60 in., or twice the normal, the amount of damage due to erosion would have been tremendous if the chaparral cover had not been there to prevent it.

DANIEL W. MEAD,[29] HON. M. AM. SOC. C. E. (by letter).[29a]—This excellent paper presents the most exact and conclusive measurements that are available of the effect of deforestation and denudation of drainage areas on the flow of streams. The only uncertainties in the conclusions as regards the streams discussed seem to be in the relative quantity and distribution of the rainfall on the areas of the Santa Anita and Fish Creeks in Southern California, and the possible different effects of such variation of rainfall on the comparative flow of these streams before and after the time when deforestation and denudation occurred. It will be noted that the only rainfall stations available for these areas are at Mount Wilson and at the Santa Anita Ranger Station, both of which are west of Fish Creek and cannot represent the rainfall accurately on that drainage area which, in each rainstorm, would probably vary somewhat from the rainfall at these stations just as the rainfalls at the stations vary one from the other. The increase in flow from the drainage area of Fish Creek, however, is so considerable that there can be no question as to the actual increase in flow resulting from the deforestation and denudation of the Fish Creek drainage area, although the actual quantity seems somewhat uncertain.

The rainfall measurements on the Wagon Wheel Gap areas are much more complete and represent the actual conditions as well as they could reasonably be determined. In the writer's opinion there can be no doubt as to the essential validity of the general conclusions that will be drawn from these data, that, in general, increased flow will follow deforestation and denudation on small drainage areas where other factors remain the same.

It is obvious that: (1) The relative results of deforestation or reforestation in each case must depend upon the physical and climatic factors that prevail on the areas from which such results obtain; (2) in applying these data to other small areas, the comparative physical and climatic factors of such areas must be considered; and (3) any variation in such factors will necessarily modify the results to be expected.

In extreme and rare cases it is possible that deforestation, and especially denudation, of small drainage areas might even reverse the results obtained under the conditions considered by the authors.

It is particularly worthy of note that the drainage areas considered in the paper are comparatively small (Area *B* of Wagon Wheel Gap, 200.4 acres; and area of Fish Creek, 6.50 sq. miles), and that while the results of deforestation and denudation are quite marked on these limited areas, it is obvious that, as affecting the larger drainage areas of which they are a part, the results might be quite insignificant and indistinguishable. Therefore,

[29] Prof., Hydr. and San Eng., Univ. of Wisconsin; Cons. Engr., Madison, Wis.

[29a] Received by the Secretary October 20, 1932.

it is still necessary to utilize inductive reasoning in discussing the application of the data from these small areas to the larger areas of great rivers.

As a basis for such reasoning, it is necessary to consider first principles as they are demonstrated by the simplest experiments and examples that are available.

Rainfall.—It is quite obvious that the principal factor that affects stream flow is the quantity and distribution of the rainfall on its drainage area. Evaporation (including plant transpiration), percolation, and run-off are dependent on the physical and climatic conditions on each drainage area. Each increases in general as rainfall increases, although not in direct proportion. The departure from direct proportion; that is, the difference between rainfall and evaporation, percolation, or run-off measures the effect of temperature, humidity, wind velocity, distribution of rainfall, and other minor factors. When the rainfall occurs in light showers, the proportion lost in evaporation and transpiration is greatly increased; and, in most places in the United States, if the annual rainfall were distributed equally over each day of the year, it is doubtful whether any of it would flow away in streams unless it accumulated as snow or ice during the winter and melted suddenly on the advance of spring. With heavy showers and torrential rainfall, run-off and percolation (on pervious soil) increase and stream flow is rapidly augmented.

Effects of Geology and Topography.—There can be no question but that, next to intensity of rainfall, the geology and topography of a drainage area are the principal factors that control the proportion of rainfall that appears as stream flow. Fundamentally, the character of bed-rock of a drainage area is important. In Wisconsin, the rainfall conditions throughout the State are fairly constant, and the average is about 31 in. An investigation made a few years ago[30] showed that the average annual run-off from crystalline rock areas was 13.09 in.; from sandstone areas, 10.27 in.; and from limestone areas, 9.39 in. In some places, these areas are deeply covered with glacial drift (200 ft., or more), so that the bed-rock effect is modified considerably by pervious and impervious mantle deposits as well as by surface slopes and surface conditions, such as forests, cut-over land, cultivated land, swamps, lakes, etc. In fact, the conditions of the mantle deposits, soils, and subsoils lying over the bed-rocks may have as great an effect on run-off—and especially on the unformity of flow—as the bed-rock.

For example, the average yearly run-off of the Manistee River, at Sherman, Mich. (1904 to 1915, inclusive), was about 17.1 in., with an average rainfall of about 29.5 in., which is 1 or 2 in. less than that for the State of Wisconsin. The bed-rock is about one-half sandstone and one-half shales, and the high average annual flow, greater than that from the crystalline area in Wisconsin, is due to the deep sandy soil and subsoil over the bed-rock that rapidly absorbs rainfall and reduces evaporation and transpiration. On account of the storage in the great bed of sand, the flow, in cubic feet per

[30] "The Effects of Bedrock on Runoff of Wisconsin Streams," by E. E. Foster, *Proceedings*, Eng. Soc. of Wisconsin, 1926, p. 162.

second per square mile, is far more uniform, with an average of 1.26, a maximum of 3.89, and a minimum of 0.67.

Any fragmentary mantle material that will remove the rain water promptly from the surface and store it at depths beyond the reach of evaporation will produce a similar effect on run-off. For example, the Deschutes River, of Oregon, has volcanic lava rock as its bed-rock. The upper part of this rock is cracked and fissured at the surface and perhaps to a depth of 1 000 ft., or more (its canyon in places is 2 000 ft. deep). It is covered by a comparatively thin and pervious soil over much of its area. From the limited data available the writer would judge that the average annual rainfall for the area probably does not exceed 12 in. The annual flow from this area averages about 8.75 in., and the flow, in cubic feet per second per square mile, has an average of 0.645, a 19-year maximum of 4.85, and a minimum of 0.38. It appears from the foregoing that the nature of the bed-rock and its pervious condition may have much to do with stream flow, but that pervious mantle deposits and fissured rock in the structure above the stream bed giving ample storage are the greatest factors in low floods and uniform flow of streams.

Surface and Soil Conditions.—Where pervious mantle deposits exist on a drainage area the disposition of the rainfall is greatly modified by the nature of the deposit and by surface conditions. A knowledge of the rainfall disposition has been of much importance in agriculture, and many experiments have been made to determine the evaporation and precolation through various classes and depths of soils and under various surface conditions. The writer has collected and analyzed the results of these experiments in another place.[31]

The points particularly noted[32] were the high evaporation and small percolation from sod, the high and maximum seepage through cultivated ground, and the high percolation and small evaporation from sand and pervious soils.

The run-off from any area is dependent on the pervious or impervious condition of the soil, the surface condition or covering, and the slope. Experiments were made in Wisconsin in 1929[33] on the run-off from various land areas. The highest average percentage of run-off found was less than 2% for forest covering, 20.1% for pastures, 24.1% for oat lands, 25.6% for corn land, and 21.5% for hay lands, the last three being probably somewhat reduced by the less average slope. With soil classed as sandy, the run-off in most cases was reduced below the average, undoubtedly by increased percolation. Here, again, the actual soil conditions are somewhat indefinite and the nature of the soil involved is not clearly discernible.

One of the most noteworthy examples of the increase in evaporation and transpiration (and possibly of percolation), and of the consequent reduction in run-off, is found in the change in the condition of Devils Lake, in North

[31] "Hydrology," by Daniel W. Mead, Hon. M. Am. Soc. C. E., McGraw-Hill Book Co., p. 138 *et seq.*

[32] *Loc. cit.*, Fig. 84, p. 139.

[33] *Bulletin 99*, Agricultural Experiment Station, Univ. of Wisconsin, Wis.

Dakota (see Fig. 10). This lake, which has no apparent outlet and is very alkaline, had an area of about 115 sq. miles as determined by the U. S. Land Survey in 1883, and was at Elevation 1 435 (corrected elevation). In 1883, the surrounding country was opened to settlement. While the average annual

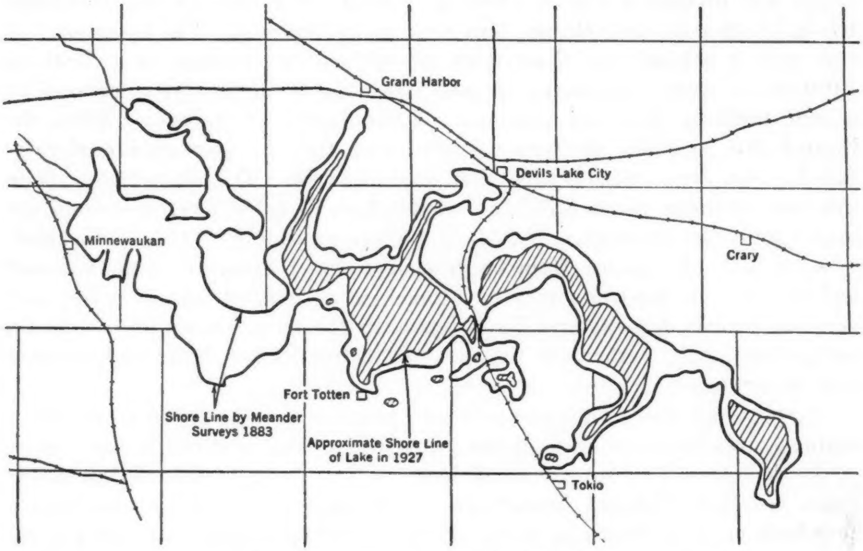

FIG. 10.—DEVIL'S LAKE, NORTH DAKOTA.

rainfall in this area has been about 18 in. (see Fig. 11), the run-off (about 1 in. per annum) from the original prairie land surrounding the lake was sufficient to maintain it at about Elevation 1 435 although it evidently fluctuated considerably with cycles of high and low rainfall years. (Note in Fig. 11 the reduction in elevation from 1 867 to 1 879). Since the land around the lake has been settled and cultivated, there has been a considerable decrease in run-off and a marked shrinkage in the height and area of the lake, evidently due to the opening of the land by cultivation. In 1927, its surface had fallen so far that it became divided into six or more smaller lakes and ponds, with an area of about 45 sq. miles and an elevation 21 ft. below that of 1883. At present (1932), the lake has fallen about 3 ft. more, or to Elevation 1 411, and has an area of approximately 40 sq. miles.[34]

A similar explanation will account for the drying up of certain springs, which has been attributed to deforestation. The probability of this assumption seems to be borne out by the data given in Table 13.[35] It will be noted from Table 13 that the daily consumption of water by most crops is given as several times greater than that of oak and fir trees. On the other hand, Harrington[35] estimates that on an average only 70% of the rainfall reaches the ground under a forest covering, and that 30% is re-evaporated from the trees and their foliage.

[34] Information furnished by E. F. Chandler, M. Am. Soc. C. E.

[35] Rept., Kansas State Board of Agriculture, December, 1889.

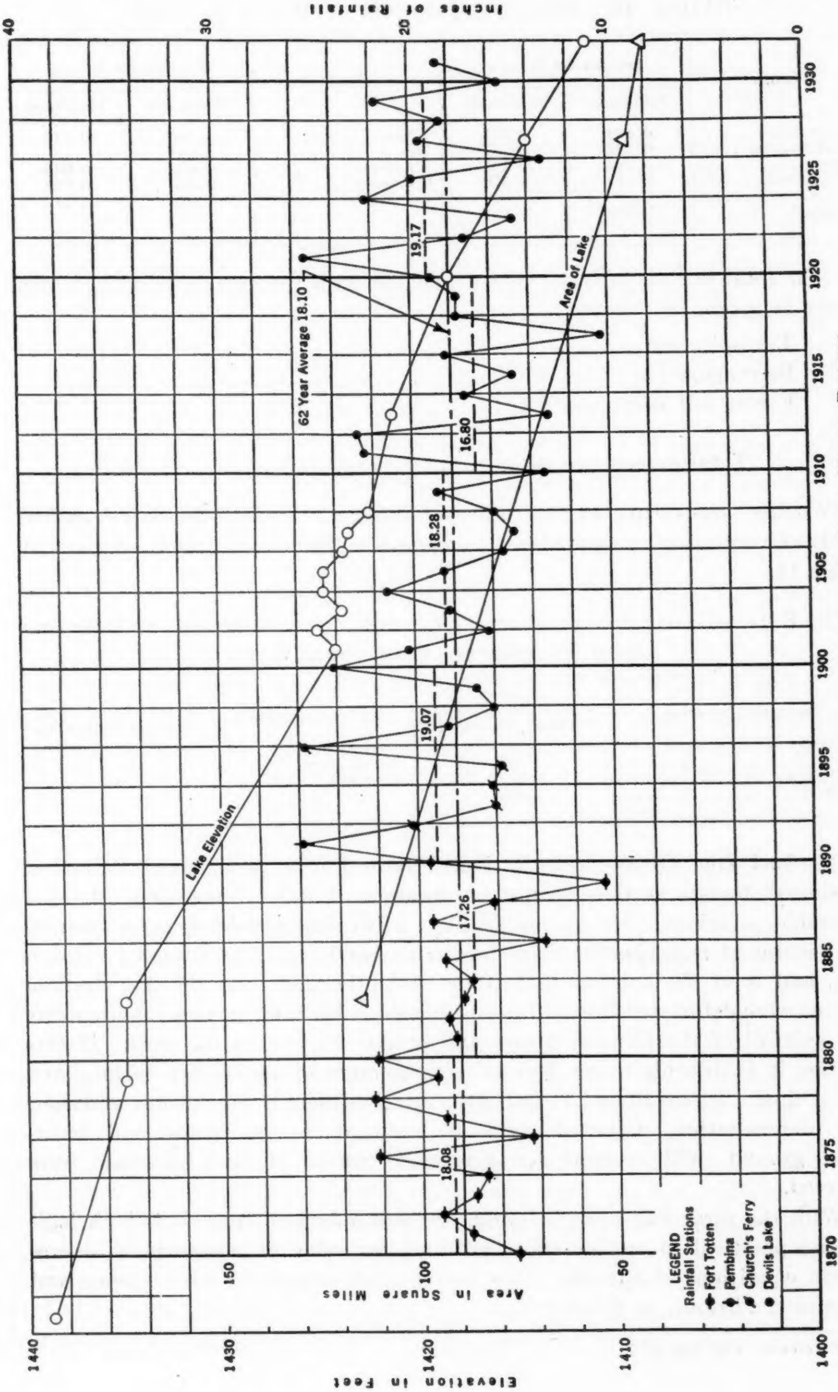

FIG. 11.—RAINFALL DATA AND LAKE LEVELS, DEVIL'S LAKE, NORTH DAKOTA.

TABLE 13.—DAILY CONSUMPTION OF WATER BY CROPS

Crops	Inches of Water		Crops	Inches of Water	
	Minimum	Maximum		Minimum	Maximum
Lucern grass	0.134	0.267	Wheat	0.106	0.110
Meadow grass	0.122	0.287	Rye	0.091	.....
Oats	0.140	0.193	Potatoes	0.038	0.055
Indian corn	0.110	1.570	Oak trees	0.030	0.038
Clover	0.140	.....	Fir trees	0.020	0.043
Vineyard	0.035	0.031			

The total use of water by trees compared with the evaporation from open water is given, as follows:

Transpiration	77%
Re-evaporation	61%
Forest soil evaporation	13%
Total percentage of open-water evaporation	151%

Woolny[35] has estimated the return of water to the atmosphere by various kinds of vegetation, compared with evaporation from open water, as given in Table 14.

TABLE 14.—PROPORTION OF WATER RETURNED TO ATMOSPHERE AS COMPARED WITH EVAPORATION FROM OPEN WATER

Vegetation covering	Proportion of water returned, percentage	Vegetation covering	Proportion of water returned, percentage
Sod	192	Mixed crops	144
Small grass	173	Bare soil	60
Forest	151		

Raphael Zon, Director of the Lake States Forest Experimental Station, considers[36] forests as the greatest evaporators of water, exceeding all other vegetable coverings. He quotes Otozky, a Russian physicist, as estimating the amount of transpiration from forests as nearly equal to annual precipitation, and is of the opinion that if the Atlantic plain and the Appalachian regions were deforested it would have a considerable influence on the humidity and rainfall of the Central States and prairie regions to the west. If this is true, it is difficult to see forests as conservers of waters for springs and stream flow. Tables 13 and 14 indicate entirely different conclusions and show that deforestation, followed with replacement by meadows and cultivated ground, will account for decreased run-off if such decrease were observed.

With the foregoing facts in mind, the writer believes that it is both logical and practical to outline the conditions favorable or necessary to assure certain conditions of run-off. This has been attempted for two extreme and opposite conditions, as follows:

[36] *Science*, Vol. 38, p. 71.

I.—Conditions Favorable to Maximum Annual Discharge and Equalized Stream Flow with a Given Rainfall:

(*a*) A fairly deep impervious basin underlying the drainage area from which no water can escape, except through the normal outlet of the stream.

(*b*) The basin in Condition (*a*) filled with pervious material, such as sand, sandstone, fractured volcanic or other cracked and fissured rocks (such rocks being covered at the surface with pervious soils), of such a nature and depth that the rains falling on the surface will sink rapidly into storage in the basin. The water must settle fast enough to avoid excessive surface evaporation and deep enough so that capillary attraction will not draw the stored water to the surface.

(*c*) The basin in Condition (*a*) with no surface storage on the drainage area.

(*d*) The basin in Condition (*a*) with only slight surface slope so that there will be no surface run-off, but rapid absorption into the pervious fill.

(*e*) A basin in which the area is entirely devoid of vegetation.

II.—Conditions Favorable to Torrential Flow and Maximum Variation in the Continuity of Flow with a Given Rainfall:

(*a*) An impervious basin, with steep channel and abrupt slopes to the divide; and,

(*b*) An impervious basin, with unobstructed channel and no mantle deposits or vegetation.

Under Conditions I, the rain falling on such a drainage area will sink rapidly into the soil and pervious underlying deposits, thus avoiding surface evaporation, and flow slowly toward the river channel, being sufficiently detained in storage to equalize the flow partly between the more or less periodic rain storms that occur over the area. Under Conditions II the rains will flow rapidly to the channel and through the river outlet in maximum floods followed by periods of entire cessation of channel flow.

In both the foregoing cases the distribution, intensity, and quantity of rainfall will have marked effect on the flow of the stream, but under the conditions of rainfall prevailing, one case will result in the best possible regulation with minimum floods and the other in maximum irregularity and maximum floods.

With these ideal conditions representing the extreme of regularity and irregularity in stream flow, almost any change in the conditions on these two areas will necessarily have opposite effects. In the first case, if the valley is covered with forests and vegetation, the pervious surface becomes more or less filled with roots and, therefore, less pervious. Evaporation will be increased, plant life will take from the supply, and flow will become more irregular and less in amount. In the second case, with the soil and detritus necessary for forest and vegetable growth, flow is obstructed, some storage is supplied, flood peaks will be reduced, and total discharge will be decreased.

Quantitative measurements of the actual effects of minor factors are frequently impossible although some effect may easily be demonstrable. Footprints across an impervious clay drainage area will fill with rain, reduce

run-off by the amount so held, and will add to evaporation and possibly to seepage. Such results are positive although indistinguishable and unmeasurable as affecting stream flow.

Perhaps a more striking example of what would seem to be an important factor as affecting stream flow is shown in the apparent lack of effect on the drainage of large areas of land on the flood flow of the stream from the drainage area. Professors Woodward and Nagler[37] found, in the case of the Des Moines and Iowa Rivers, where a large portion of the drainage areas had been artifically drained, that during flood periods there were no significant changes in the behavior of the rivers that might be attributed to drainage.

The hydraulic engineer is perhaps more deeply interested in the question under discussion than are the men of almost any other profession. The conservation of water for water supply, water power, navigation, irrigation, etc., and the question of maximum floods in drainage and flood protection works are of the greatest importance. If water can be conserved, supplies increased, and floods prevented by any reasonable amount of forest covering, it is most important for him to know and to appreciate such facts.

Numerous investigations have been made by engineers interested in this question and unbiased in their opinion, and some of the results of such investigations are worthy of brief mention in this connection. Perhaps the most noteworthy of these studies was that of the late H. M. Chittenden, M. Am. Soc. C. E., already referred to by the authors.

In 1910, the writer endeavored to determine what influence, if any, the cutting of Wisconsin forests had had on the flow of Wisconsin streams.[38] No long-time measurements of stream flow were available, but from the gauge-height measurements of the U. S. Weather Bureau and the U. S. Army Engineers, the annual and progressive means of the annuals were found for the Wisconsin and Upper Mississippi Rivers.

The diagrams[39] clearly showed that high, mean, and low waters followed the quantity and distribution of the rainfall, and that there had been no marked change in the normal height of floods or in extreme low-water elevation.

In an investigation of the relation of forests to stream flow on the Tennessee and Cumberland Rivers, W. W. Harts, Brig.-Gen., U. S. A., M. Am. Soc. C. E., found[40] the relations of the height of high and low water at various periods during and following deforestation to be as shown in Table 15. He estimated that, in 1909, 60% of the drainage area of the Tennessee River was forested and that 20% of the area had been cleared in the preceding twenty-five years.

While these data (Table 15) might be taken to indicate a greater prevalence of extreme high and extreme low water during the earlier period when

[37] "The Effect of Agricultural Drainage upon Flood Run-Off," by Sherman M. Woodward and Floyd A. Nagler, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 821.

[38] "The Flow of Streams and the Factors That Modify It," by Daniel W. Mead, Hon. M. Am. Soc. C. E., *Bulletin No. 425*, Univ. of Wisconsin.

[39] *Loc. cit.*, Figs. 68 to 70, p. 308.

[40] *Professional Memoirs*, U. S. Corps of Engrs., Vol. I, 1909, S. P. 937, p. 397.

a greater proportion of forest covering existed on the drainage area, General Harts stated that the frequency and height of high water in both these rivers, as well as the duration of low water, followed the rule of precipitation closely enough during the period under observation to force the conclusions that this is the principal cause of the variation in these streams, and that "the introduction of forestry as an important factor in either direction is forced and illogical."

TABLE 15.—Relations of the Height of High and Low Water During and Following Deforestation

Period	Tennessee River		Cumberland River		
	Gauge height, in feet	Average number of days per year	Gauge height, in feet	Total number of days	Average number of days per year
High-Water Stages					
1899–1908...........	>25	7.0	>35	78	.....
1889–1898...........	>25	7.1	>35	123	.....
1879–1888...........	>25	11.6	>35	178	.....
Low-Water Stages					
1899–1904...........	<1	10.3	<1	.....	20.8
1889–1898...........	<1	7.7	<1	.....	32.0
1879–1888...........	<1	20.4	<1	.....	32.5
1899–1904...........	.....	.....	<5	.....	161.0
1889–1898...........	.....	.....	<5	.....	153.8
1879–1888...........	.....	.....	<5	.....	141.8

The investigations by General Harts were based on areas, in which deforestation had taken place through the cutting of timber through a considerable term of years.

Edward Burr, Col., Corps of Engineers U. S. A., (*Retired*), M. Am. Soc. C. E., investigated the result of reforestation in the Merrimac River Basin on the flow of that river.[41] In regard to the forest conditions, Colonel Burr states that (1) deforestation continued progressively from the earliest settlement to about 1860-70; (2) reforestation through natural processes progressed from 1870 to 1910 at a rate that has increased from 1880 to 1900, or later; and (3) forest areas were larger in 1910 than in 1860-1870 by as much as 25 to 30% of the entire basin. Perhaps better data (longer and more accurate) existed for this area as regards rainfall and stream flow than on any other for which similar studies have been made.

These studies, which seem to have been very thorough, included investigation of the relation of precipitation to high, mean, and low-water conditions. As in other studies, the irregularity of the consecutive annual occurrence of rainfall and run-off is so great that progressive averages are needed in order to ascertain tendencies in their relations, and, in these cases, 10-year progressive averages were used. Colonel Burr calls attention to the fact that his

[41] See House Doc. No. 7, 62d Cong., First Session, May 23, 1919; also, *Engineering News*, Vol. 66, p. 100 *et seq.*

diagrams for total annual days of flood and for days of flood per inch of precipitation are approximately parallel to the precipitation diagram, indicating quite clearly that, when eccentricities are eliminated by averages over a period of 10 years or more, the duration of floods is dependent largely upon precipitation and that the duration of floods per inch of precipitation increases with the latter. Furthermore, he stated that the same characteristics are recognizable in regard to annual discharge, annual flood discharge, the percentage of annual discharge that runs off during flood stages, and the flood discharge per inch of precipitation. These characteristics are so marked that it is difficult if not impracticable to eliminate the influence of precipitation and to ascertain the existence and effect of any minor influence, such as variable forest conditions.

Colonel Burr found no evidence that deforestation or reforestation had modified the stream flow of the Merrimac River in any way. In this connection it seems pertinent to call attention to the fact that the highest floods that have ever been recorded on the Mississippi River, at St. Louis, Mo., were in 1785 and 1844, long before deforestation could have had any effect on the flow of that river.

These investigations and the data considered point to the conclusion that: (*a*) On large drainage areas any acts of man except the construction of relatively large storage reservoirs or extensive flood protection works, are so limited in extent that they can have little influence on the flow of streams; (*b*) stream flow follows rainfall; and (*c*), large floods are due to torrential rains, together with other physical and climatic conditions favorable to flood conditions. On small areas the cultivation of the land and the existence of extensive forests may have a considerable effect in decreasing the flow of small streams draining such areas, except that such effects are so small that they are likely to become insignificant and indeterminate on the flow of the main streams to which these small streams are tributary.

In, general, there can be no doubt that all the factors described in this discussion have some effect on the disposition of rainfall, and, therefore, on run-off, but unless the effect is determinable and measurable it is theoretical and of little practical importance.

This subject has been before the public for many years and has been so surrounded by tradition and superstition that it has become greatly obscured. Extravagant claims as to the value of forests as a means of flood protection still persist. The lengthy discussion of this subject grows wearisome and would seem to be quite unnecessary were it not for its possible influence on legislation and the possibility of resulting large and useless (as far as flood protection is concerned) public expenditures. As late as February, 1929, the then Secretary of Agriculture, in his letter of transmittal to the President of the report on "Relations of Forestry to the Control of Floods in the Mississippi Valley," states:[42]

"Added study * * * shows that the forests of the Mississippi watershed were responsible for a reduction in the possible flood crest of nearly 15 inches.

[42] House Doc. No. 573, 70th Cong., Second Session, pp. III and IV.

Furthermore were all the forests of the Mississippi valley properly protected and managed in accordance with established forestry principles and practice a further reduction in possible flood crest of 55 inches would be possible."

He further states that this retaining effect would be equal to a reservoir storage of 46 000 000 acre-ft.

Such statements are so palpably absurd that they might be ignored were it not for the fact that they form the basis for a proposed expenditure of about $2 000 000 per year for protection and extension (?) of the forests in the Mississippi Basin.

RALPH R. RANDELL,[43] M. AM. SOC. C. E. (by letter).[43a]—On the only two water-sheds for which the matter has been investigated conclusively, it was found that: (1) Forests and brush substantially lessen stream flow at practically all seasons and stages, including flood, minimum, total, and average flows; and (2) forests and brush substantially lessen erosion.

These relationships have been established experimentally as to Area *B* of the Wagon Wheel Gap experiment, in Colorado, and the Fish Creek water-shed, in Southern California. They were demonstrated during each of seven consecutive years in the case of the Wagon Wheel Gap water-shed, and six in the case of Fish Creek, a total of thirteen years. No year showed contrary results with either water-shed.

The Colorado and Southern California water-sheds are widely different geographically, climatically, topographically, geologically, and in soil and vegetative cover, form of precipitation, and regimen of run-off. Especially significant is the fact that the winter precipitation on the Wagon Wheel Gap water-shed falls in the form of snow, remains frozen on the ground all winter, and does not run off as stream flow until melted by the rising temperatures of the ensuing spring. The precipitation over the Fish Creek water-shed, on the other hand, falls as rain, and runs off immediately, except in so far as it evaporates or seeps underground temporarily.

The fact that all conclusive investigations show the foregoing relationships, even under such widely different circumstances and over such long periods of time, presumptively establishes that these relationships are of general application, and are not merely the result of any local or temporary peculiarities.

The same laws necessarily apply also to large water-sheds, as well as small ones; because large water-sheds are only combinations of small ones, and, therefore, must inevitably re-act to given influences, such as vegetative cover, in the same manner as the small ones of which they are composed.

The authors' contribution consists of the presentation and original analysis of the Fish Creek data, and a re-analysis of the Wagon Wheel Gap data along lines somewhat different from those used by Messrs. Bates and Henry,[7] but with the same results. Their excellent paper should be of great benefit

[43] Senior Engr., Federal Power Comm., Washington, D. C.

[43a] Received by the Secretary October 21, 1932.

[7] *Proceedings*, Am. Soc. C. E., August, 1932, p. 1039.

in directing attention to the real facts as to the influence of forests on stream flow. It is high time for engineers to discard, once and for all, the hoary fallacies that have been propagandized so widely and persistently during the last score or more of years. Especially important is it that they realize at last that all conclusive investigations show that forests and brush do not increase, but instead considerably lessen, both total water supply and low-season stream flow.

H. K. BARROWS,[44] M. AM. SOC. C. E. (by letter).[44a]—A paper such as this is always welcome, adding as it does to the present information on a somewhat controversial subject.

The writer has studied the data presented from the viewpoint of water losses. It is obvious from Fig. 12 that on both the Wagon Wheel Gap and

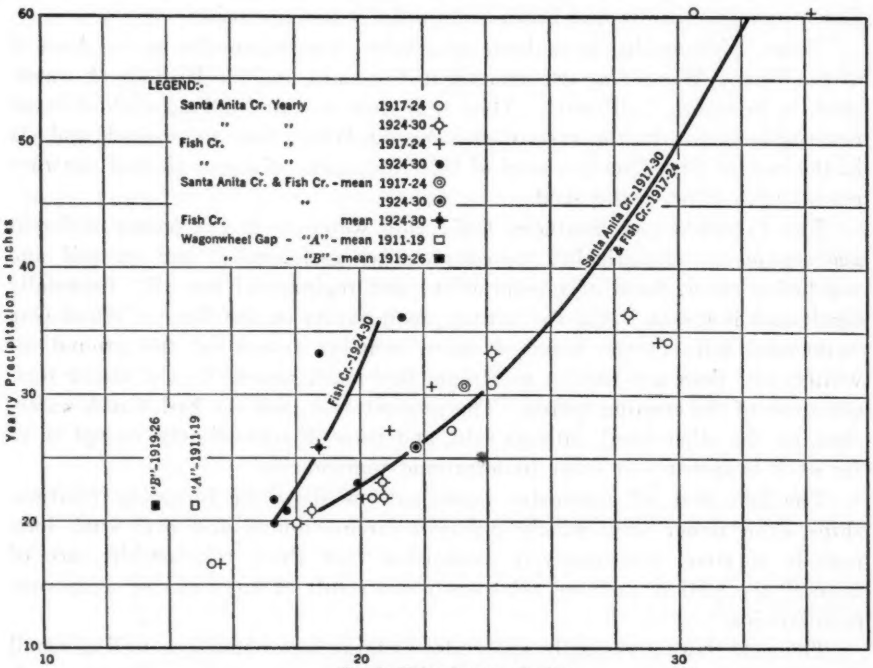

FIG. 12.—COMPARISON OF PRECIPITATION AND WATER LOSSES.

Fish Creek drainage areas the yearly water losses have been decreased on the deforested or burned areas, and, hence, the run-off of these areas—which is the precipitation minus water losses—has been increased.

The lessening of water losses in respect to precipitation for the Southern California area is approximately as shown in Table 16.

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[44] Prof. of Hydr. Eng., Mass. Inst. Tech.; Cons. Engr., Boston, Mass.

[44a] Received by the Secretary October 28, 1932.

The Wagon Wheel Gap observations cover a much narrower range of precipitation (from about 17 to 24 in. yearly), but the period means for about 21 in. of rainfall, shown in Table 16, indicate fairly consistent results.

TABLE 16.—WATER LOSSES

Yearly precipitation, in inches (1)	Water Losses, in Inches: Before deforestation (2)	Water Losses, in Inches: After deforestation (3)	Column (2) minus Column (3), in inches (4)	Ratio: Column (4) / Column (2) (5)
Southern California Areas				
20	18	17+	1	0.05
30	24	20	4	0.17
40	27	21	6	0.22
Colorado Areas				
21	15	13.5	1.5	0.10

It is evident that, in many cases, the lesser soil evaporation upon forested areas is more than offset by the demands of the forest for transpiration as well as its interception losses. The net result is that the forested area may show greater water losses and, hence, less yearly run-off than the deforested area, under conditions otherwise similar.

There is much misunderstanding on this matter by the layman, and many water-works officials in the Northeastern United States are devoting time and money to tree planting on drainage areas used for water supply, with the idea that they are thus increasing the supply, when they are more probably diminishing it. It may be worth while for other reasons, and perhaps a good investment, to adopt such programs of tree planting, but certainly as regards increasing the water supply the results are likely to be deterimental rather than helpful.

More and better information is needed on this subject in other sections of the country and, if possible, some experimental work in the study of comparative water losses on forested and open areas.

DONALD M. BAKER,[45] M. AM. SOC. C. E. (by letter).[45a]—The basic equation covering the disposition of precipitation is as follows:

$$\text{Precipitation} = \text{Evaporation} + \text{Transpiration} + \text{Surficial Run-Off} + \text{Ground Storage}$$

In a closed water-shed, ground storage usually appears as stream flow. Any physical change on a water-shed affecting either the right or left-hand part of this equation will likewise affect the other part. Precipitation that reaches the ground surface might be increased by deforestation through the elimination of interception by tree or brush growth. The extent of this interception, however, is small, except in light showers or snowfall, and is probably balanced or equalized by greater evaporation from the ground surface.

[45] Cons. Engr., Los Angeles, Calif.

[45a] Received by the Secretary October 28, 1932.

Assuming little or no increase in precipitation, increase in surficial run-off, and ground storage must mean of necessity that evaporation and transpiration, or at least the algebraic sum of these two factors, are decreased.

The observations in the two localities show the following results for the second period (depths of run-off over the water-shed being given in inches):

	Wagonwheel Gap, Area *B*	Fish Creek
Actual	7.261	6.95
Normal	6.306	5.40
Difference	0.955	1.55
Percentage increase over normal	15.2	28.7

Transpiration by forest or brush cover certainly is greater than 1.0 to 1.5 in.—the increase in stream flow due to deforestation—and it may reasonably be concluded that a considerable portion of the decrease in transpiration following the removal of trees and brush is counterbalanced by increased evaporation from a warmer soil.

The fact that the two localities at which the observations were made had widely differing climatic characteristics makes the analysis of results of greater than ordinary interest. Wagonwheel Gap is at a high elevation, has a low mean annual temperature of 34° Fahr., with a range of temperature from below zero to 80° Fahr., and a relatively uniform seasonal distribution of precipitation, a considerable portion of it in the form of snow. Surficial run-off in Wagonwheel Gap is small, except when the snow is melting, most of the stream flow coming from ground storage, which, however, is receiving constant replenishment. The percentage of run-off to precipitation under normal conditions was from 29 to 30, which was increased after deforestation to 34.3. Fish Creek drains a much lower area, has a much higher mean annual temperature, probably about 60° Fahr., with, however, a smaller annual range (from below 32° to 100° Fahr. at Mount Wilson) of precipitation concentrated in a few months. Surficial run-off is large and little of the stream flow comes from ground storage.

An analysis of the increase of the actual run-off over the normal for both Wagonwheel Gap, Area *B*, and Fish Creek, for the period after deforestation, as presented in Table 17, is of interest.

The increase in actual over normal run-off appears to follow a different rule on each water-shed. At Wagonwheel Gap during November, December, January, and February, when the weather is very cold and snow is probably accumulating, the proportionate increase is small, increasing during the months of melting snow. Again, in June, there is an actual decrease, while in July and August the increase is small. The increase from November to May is probably due to faster melting of the snow, which, in turn, is caused by higher temperatures due to lack of shade from forest cover, this rate of melting being accentuated as the general temperature increases during March, April, and May. In June, soil evaporation is in excess of decreased transpiration, but during July and August it does not exceed the saving made through lack of transpiration.

On Fish Creek, the situation is different. Precipitation ceases in April or May, and stream flow is entirely supplied from ground storage from that time until November or December. The increase during the rainy season,

TABLE 17.—ANALYSIS OF INCREASE IN ACTUAL RUN-OFF OVER NORMAL RUN-OFF AFTER DEFORESTATION

Month	Wagonwheel Gap			Percentage of increase	Fish Creek			Percentage of increase
	Actual run-off, in inches	Normal run-off, in inches	Difference, in inches		Actual run-off, in inches	Normal run-off, in inches	Difference, in inches	
October	0.363	0.329	0.034	10.3	0.06	0.01	0.05	500
November	0.342	0.326	0.016	4.9	0.16	0.08	0.08	100
December	0.328	0.316	0.012	3.8	0.28	0.17	0.11	64.7
January	0.313	0.302	0.011	3.6	0.32	0.22	0.10	45.5
February	0.287	0.274	0.013	4.8	1.93	1.57	0.36	22.9
March	0.387	0.317	0.070	22.1	0.92	0.72	0.20	27.8
April	0.699	0.494	0.205	41.6	2.52	2.15	0.37	17.2
May	2.699	2.140	0.559	26.1	0.40	0.34	0.06	17.6
June	0.791	0.852	—0.061	—7.2	0.19	0.11	0.08	72.7
July	0.400	0.375	0.025	6.7	0.08	0.02	0.06	300
August	0.335	0.306	0.029	9.5	0.05	0.01	0.04	400
September	0.317	0.275	0.042	15.3	0.04	0.00	0.04	∞
Total	7.261	6.306	0.955	15.2	6.95	5.40	1.55	28.6

due to increased surficial run-off which, in turn, is due to lessened ground absorption, is small. Ground storage, which has accumulated during the rainy season, and which before deforestation supplied the brush cover and was transpired, now appears as stream flow.

A number of other things of interest are noted in the Fish Creek observations. In Tables 6 and 7 the effect of development within a short time of new cover on the reduction of maximum daily discharge, maximum peak discharge, and ratio of peak to daily discharge, indicates that it is grass and small vegetal cover that reduce surficial run-off, rather than heavy cover. The large increase in summer flow immediately after the fire on Fish Creek is probably due to the fact that increased erosion filled up the canyon with detritus and afforded an increased opportunity for ground storage, and that such fill was either eroded away, or through increased vegetation the water in storage was depleted through transpiration.

The analyses of the Wagonwheel Gap observations and of the Fish Creek data, as given in the paper, are distinct contributions to engineering literature, but broad conclusions cannot be drawn from them until much more quantitative data are obtained from other areas with different climatic, geological, and topographic characteristics.

Southern California has a marine type of climate. Moisture-laden air moves inland from the ocean, is forced to rise when it strikes the Sierra Madre Mountains, and in cooling, loses its moisture in the form of precipitation. The lack of summer rains is due to the fact that sufficient cooling does not occur during that period. Deforestation might be of value on a large scale under such climatic characteristics, provided that flood flows were not increased to a damaging extent, as it would not affect the source of supply for precipitation.

In a section with a continental type of precipitation, a considerable part of the moisture supply for precipitation somes from local evaporation and transpiration. Assuming an area several thousand square miles in extent (which had this type of precipitation source), to be deforested, the net evaporation plus transpiration would undoubtedly be reduced. Since it is the moisture received from these sources that goes to make up at least a part of the precipitation, the latter would likewise be reduced with a consequent reduction in surficial run-off and ground storage until a new state of equilibrium is established. As a specific example, consider conditions in the Southeastern States, where precipitation ranges from 50 to 70 in., stream flow from 20 to 40 in., and losses, which are made up of evaporation and transpiration, are constant at about 30 in., with none of the precipitation, or little of it, in the form of snow.

Deforestation would increase surficial run-off immediately following precipitation and undoubtedly would decrease ground storage. Whether the net change in surficial run-off plus ground storage would be plus or minus has not been established quantitatively, but the probabilities are that it would be a minus change. According to the observations in question, the change in evaporation plus transpiration would be minus, and since the equation of disposition of precipitation must balance, a reduction in precipitation undoubtedly would ensue. Since a large proportion of the precipitation during the growing season in this section is derived from local sources of moisture, the loss would be concentrated during this period, and might have a serious effect upon agriculture.

The foregoing discussion is given with a full realization that it is not based on any quantitative background, but with the purpose of showing that the conclusions reached from the observations in question cannot be accepted as general under all topographic, climatic, and geological conditions until many more quantitative data, collected under a wide variety of differing conditions, are available for analysis. It is hoped that this paper will result in the initiation of other observations of a like character in other places.

The methods used by the authors in the analysis of these observations are sound, and the conclusions drawn, as far as they apply to the water-sheds in question, are without question correct.

RALPH A. SMEAD,[46] ASSOC. M. AM. SOC. C. E. (by letter).[46a]—The value of the analysis offered by the authors should be evident to any one who has given thought to the question of forests and stream flow. The writer was engaged with the U. S. Geological Survey some years ago in an earlier study of this problem under much less favorable conditions, and is doubly appreciative of the unusual opportunities for study offered by the areas analyzed in this paper. In 1911 and 1912, the Geological Survey conducted an investigation of a group of small water-sheds in the White Mountains of New Hampshire. As the field data covered only a little more than a year it was impossible to compare forested with deforested conditions on a single water-shed, and evi-

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[46] Structural Engr., Los Angeles, Calif.

[46a] Received by the Secretary October 28, 1932.

dence had to be drawn from stream flow and precipitation records made simultaneously on the various water-sheds. Some of these water-sheds were covered with virgin forests, some were deforested by cutting, and some were denuded by forest fires. One great difficulty arose from the presence of numerous varying elements, other than forest cover, that might affect stream flow. Such elements include steepness of slope, exposure to sun and wind, size of water-shed, nature of soil, method of forest removal (whether by cutting or by fire), length of time since removal of forest, amount and nature of precipitation, etc. Even to estimate the effect on stream flow of such elements, and to determine the effect of forest cover alone, was far from a simple problem.

In the case of the Wagon Wheel Gap and Southern California areas, on the other hand, such varying elements either were not present or could be eliminated effectively, due to the fact that a single water-shed could be studied both with and without forest cover, and due also to the presence of a closely similar neighboring water-shed by means of which the variations of meteorological conditions from year to year could be taken into account in determining the "normal" discharges (as the authors term them) for the period after forest removal. Thus, from these unusually adaptable areas, data have been furnished, and an analysis has been made that presents convincing conclusions as to the actual effect of forests on stream flow.

The authors call attention to certain variations in the run-off relations between Areas *A* and *B* of the Wagon Wheel Gap section, and take those variations into account, probably by as exact a method as is practicable, in computing the "normal" discharge from Area *B*. It would be of interest, however, to investigate the causes for those variations, with a view to a more accurate determination of the "normal" discharge. For instance, the lag of several days between the spring peak discharges of Areas *A* and *B* before deforestation is apparently due to the difference in exposure to sunshine on the two areas, and the consequent delay in snow melting on Area *B*. This is indicated by the difference in direction of the slopes, by the disappearance of the lag at the time of the rain storm of October 5, 1911, and by the decrease in the lag between these peaks after deforestation had given the sun better access to Area *B*. It is possible that the relationships between cloudiness (if records exist), and the variation in amount of the lag would further establish sun exposure as the cause of this lag. Cloudiness records, then, for the period subsequent to deforestation would make possible even more accurate determinations of the "normal" discharges for this period.

The writer appreciates the careful selection and orderly arrangement of data as presented in the various tabulations and charts in the paper. The authors have confined themselves to the essentials of the problem in the interests of clearness and simplicity. However, the writer believes that one more chart, similar to that of Fig. 7, but covering an entire year of Area *B* at Wagon Wheel Gap, would be worth while. It would show clearly and at a glance, as only a chart can show, the distribution, for the entire year, of the increase in annual run-off. It would also show many other main features of the analysis.

GEORGE H. CECIL,[47] ESQ. (by letter).[47a]—The subject of this paper is of vital importance to Southern California. Perhaps nowhere else in the world, and certainly nowhere else in the United States, is water of such vital importance. While it is necessary to life everywhere, in a semi-desert region it is essentially the determining factor in urban and industrial growth and agricultural development.

From the standpoint of the value of crop production, the Valley of Southern California is the richest agricultural area in the United States. Already, in some localities, agricultural development has progressed beyond the assured long-time average yield of the local water resources. Given a cheap and adequate supply, many more acres of land are susceptible of development.

The importation of water into Southern California is possible and, in the case of Los Angeles, with its supply furnished in part from the ranges of the Sierras, it is an accomplished fact. Further importations from the Colorado River are contemplated, and plans for furnishing this supply to the Metropolitan Area are well advanced. These imported waters, however, will be needed eventually for urban supply and, even if they were sufficiently bountiful to permit their use for agriculture, the cost is prohibitive for any except the most valuable lands.

The Federal Government, the State, counties, and many of the municipalities of the region combined expend annually well over \$1 000 000 in the protection from fire of the mountain areas of the region. This vast sum is expended not for the purpose of protecting a commercial timber supply or, for the most part, even a growth of tree species. The cover is, in the main, chaparral—a word derived from the Spanish and signifying a growth of dwarf oak. Trees are found in the canyon bottoms, at the higher elevations, and scattered here and there along the ridges of the chaparral covered slopes. There is ample evidence that these tree species at some time in the past comprised a more appreciable part of the cover, but at the present time by far the greater part of these mountain water-sheds is covered with a heavy growth of brush species, so dense (except where recently destroyed by fire), as to be impenetrable except on foot, and even then with the greatest difficulty.

Protection is afforded the cover on these areas on the assumption that: (1) It is a conserver of water; (2) it controls erosion; (3) it retards flood run-off; and (4) recreational value would be impaired by its removal.

If the authors are correct in their conclusions, the agencies now expending such vast sums in protection might well pause and consider the justification of these expenditures. These conclusions are, that such protection is not justified from the standpoint of the first objective; that the second is of little importance; the third, while the relationship is admitted in part, is apparently not considered of sufficient importance to affect the conclusions; and the same is, by inference, the case with regard to recreation since, although this form of use is mentioned, the conclusion reached is that damage by fire does not interfere with the recreational use of an area. Certainly, if all the fac-

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[47] Executive Secy., Conservation Assoc. of Los Angeles County, Los Angeles, Calif.

[47a] Received by the Secretary November 16, 1932.

tors of the situation have been given proper consideration and the conclusions are correct, the Federal Government, State, and local agencies would not be justified in the expenditures they are now making.

There are several factors, however, that are important to the water user in Southern California, that have been overlooked entirely, and others that have not been given their proper value in arriving at a conclusion.

The mountain areas of Southern California perform their highest function in the production of a usable water supply for the municipalities and agricultural areas dependent upon them. Their use for recreation may be considered as of secondary importance.

The prime requisite in water production is that the water must be usable. This factor is of greater importance than the quantity produced, and is vastly more important than a minor increase in the sustained summer flow. Probably 95% of the water used for domestic and industrial purposes, outside the City of Los Angeles itself, is pumped, as is also upward of 80% of that used for irrigation. The reasons for this are several. The mountain canyons are steep and sites suitable for reservoirs are limited in number and expensive of development, precluding the storage of water except for municipal use where large communities are concerned. Further, the entire mountain area is faulted so badly and broken into blocks that the number of sites at which dams may be constructed safely, is few.

The mountain ranges of Southern California are the result of an upthrust of the earth's crust. The same, and subsequent, movements that gave birth to these mountain ranges and broke them into blocks, caused that part of the region lying between the foot of the upthrust and the Pacific Ocean to drop, thereby creating vast basins which, in the course of the ages, have been filled with detrital matter brought down from the mountains. This coastal plain is further characterized by the existence of lesser faults, resulting in the creation of numerous minor basins. In a study of the underground water of the South Coastal Basin, the State Division of Water Resources recognized no less than thirty-one such underground basins or reservoirs. The replenishment of these underground reservoirs which, as indicated, furnish the water supply for the major part of both urban and agricultural development, is of paramount importance.

In order that the water finding its way from the mountain areas on to the coastal plain may perform its maximum of use, as much of it as possible should percolate into the underground strata near the mouths of the canyons from which it issues. For years past, several communities, represented by the water companies supplying them, have spread the flood waters over the detrital cones by means of lateral ditches, increasing the wetted area and materially increasing percolation over that obtaining under natural conditions. The experience of these companies has proved beyond a doubt that, in order that water may be spread successfully and the maximum of percolation secured, it must be free of suspended matter. It is often necessary, during the first run-off of the season, to by-pass to the ocean a varying part of the flood flow. In the case of a water-shed that has been run over by fire, the quantity that must be by-passed because of the silt load is many times

as great as that under normal conditions. It is admitted by the authors that the flood flows for the first few years after a fire carry an added quantity of silt.

Since repeated burning will be necessary to secure the increased flow that theoretically will result from destruction of the cover, this added silt load may be expected to continue until all erodable material has been carried off. Under such conditions, the water that will be lost to the ocean instead of being percolated into the gravels is many times the quantity that could possibly result from the added summer flow indicated. In fact, careful analysis of the estimated increase in the summer flow will show that although in percentage it reaches a startling figure, when reduced to actual run-off, in second-feet, it is almost negligible. Further, if a proper allowance is made for the solid material that will be carried at flood stage, and to some extent continuously, the increase in run-off for the winter or flood season will be decreased materially.

The quantity of eroded material carried by the floods from Fish Canyon is given little weight by the authors in arriving at their conclusions. The truth is that the detrital matter carried by Fish Creek, Sawpit Canyon, and Rogers Canyon, as an aftermath of the 1924 fire, not only damaged and destroyed a considerable area of agricultural land, but cost thousands of dollars for removal from highways and the railroad along the base of the mountains.

No flood-control dams have been constructed in these minor canyons. However, the Los Angeles County Flood Control District is spending approximately \$25 000 000 in the construction of dams in other and more improtant water-sheds. The importance of water-shed cover to these engineering projects is freely recognized by engineers of the District. In a report on check dams dated May 22, 1931, its Chief Engineer, E. C. Eaton, M. Am. Soc. C. E., gives as one of his conclusions the following: "The most effective agent in both the regulation of débris and the reduction of flood peaks is the vegetation covering the watershed." Again, in a paper entitled "The Fire Problem and Erosion Prevention in Relation to Flood Control—Los Angeles County Flood Control District," presented at the Rural Fire Institute in February, 1931, Mr. Eaton states:

"I have no figures on what fire prevention costs per year per square mile of watershed, but damages from débris deposits after fires, followed by only moderate storms, commonly run from \$25 000 to \$50 000 per square mile, and protective measures run anywhere from \$2 000 to \$20 000 per square mile. It is certain that prevention is much the cheaper."

In fact, the deductions made by Messrs. Hoyt and Troxell, if carried to their logical conclusion, would defeat the objectives sought in increased summer flow. With the destruction of the water-shed cover, the loose material would be carried eventually by the torrential rains down the steep canyons on to the valleys below. Even if the resulting damage could be ignored, as is suggested, the materials in which must be stored the water to produce the increased summer flow indicated, would themselves be carried away eventually. The result would be that the entire winter precipitation would immediately find its way to the valleys, and summer flow would be nil.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

GEORGE WASHINGTON BRIDGE: ORGANIZATION, CONSTRUCTION PROCEDURE, AND CONTRACT PROVISIONS

Discussion

BY J. P. CARLIN, M. AM. SOC. C. E.

J. P. CARLIN,[6] M. AM. SOC. C. E. (by letter).[6a]—As a principal in two of the contracts in connection with the George Washington Bridge, with its resulting association with the various divisions of the Port Authority engineering organizations, the writer feels, after carefully reading Mr. Stearn's paper, that nothing can be added thereto without going into details, apparently omitted for the sake of brevity.

A part of the paper under "Form of Contract" relates to a subject of great importance in construction work where adjoining properties might be affected by contract operations. Mr. Stearns states that full responsibility was placed upon the contractor for his work and for all damages and claims for damages resulting from it. He states also that the contractor was the insurer of the Port Authority against all contingencies arising out of the performance of the work, and that this course was justified because of the thorough studies, surveys, and sub-surface investigations that preceded the issuing of the contract documents, and because the complete and detailed drawings and specifications reduced to a minimum the uncertainties faced by the contractor.

Frequently property owners take advantage of this blanket indemnity running in favor of the other party to the contract, and make claims against the contractor for every conceivable damage that may have occurred to their property during the course of contract operations. Recently, a particularly voracious owner presented claims for repairs alleged to have been caused by a contractor's operations, whereas the repairs were made the year prior to the signing of the contract. Another case was that of a tenant in one of the apartment houses adjacent to a job, who complained that blasting operations had damaged his radio set; whereas there were twenty-five other tenants in the same building, none of whom made a similar claim, and all of whom had radio sets. Frequently, the property owner will claim damages

NOTE.—The paper by Edward W. Stearns, M. Am. Soc. C. E., was published in October, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[6] Pres., Cornell Contracting Corp., New York, N. Y.

[6a] Received by the Secretary November 16, 1932.

to a sidewalk at points as much as 100 ft. away from the work, where construction operations could not possibly have affected it, and where observation disclosed that coal trucks and moving vans had actually caused the damage. Still another claimed damages to his roof, alleging that it was caused by the contractor's operations, and upon investigation it was found that the roof had been placed sixteen years previously, and that the galvanized gutters and flashings were completely corroded. All these cases confirm a statement[7] made by Miles I. Killmer, M. Am. Soc. C. E., who points out that in many instances some owners will seize upon the occasion of the presence of a contractor to bring suit even when the physical damages are so slight as to require magnifying glasses to be seen.

These instances illustrate the importance of making a preliminary examination, and show that the contract should require such a survey to be made of all houses and property along the line, or adjacent to the proposed construction. If included in the specifications of the proposed work, it would eliminate all doubt as to the obligation of the owner of the improvement to establish a record of the existing conditions prior to the commencement of the work by the contractor, and to which record the property owner should be obliged to subscribe, together with the successful contractor.

Another interesting point is the matter of liquidated damages for failure to complete the work on time, and which clause has quite generally displaced the bonus and penalty agreement which was common at or about the time of the World War. If the owner of an improvement is injured by its delay in completion, it follows that he must benefit if the work is completed earlier than the contract time. Furthermore, a public improvement is an economic necessity and the sooner it is completed, the quicker it will function and fulfill the necessities of the public. If, therefore, both the owner and the public are benefited by earlier completion, an incentive should be offered to the contractor to partake of a portion of this benefit which, in effect, would be a part of the owner's carrying charges, and which would otherwise be wasted.

It is erroneous to assume that the entire bonus for earlier completion represents profit, because the contractor spends a part of this bonus for the purchase of additional equipment, and absorbs the resulting inefficiency which follows from speeding up the work. The bonus clause is of great advantage to the owner if the work is properly co-ordinated because it assures earlier completion than by any other clause, such as that of liquidated damages.

Where several independent contracts are involved, as was the case on the George Washington Bridge, the writer believes, with Mr. Stearns, that a bonus arrangement would not have redounded to the advantage of the Port of New York Authority. It is rare, however, that such perfect co-ordination of all the reciprocating parts of an enterprise of the magnitude of the George Washington Bridge can be as nicely calculated, and anticipated, as was the case on this bridge. The result is an outstanding tribute to the skill and efficiency of the engineering organization of the Port of New York Authority, and it evidences co-operation on the part of all the contractors engaged on this work.

[7] *Proceedings*, Am. Soc. C. E., October, 1932, p. 1392.

PROCEEDINGS

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PAPERS AND DISCUSSIONS

VOL. LVIII

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SUBJECT INDEX

AUTHOR INDEX

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibilty. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from December 15, 1932.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	*20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

BAKER, WARREN FRANCIS, Newton Centre, Mass. (Age 33). Structural Draftsman, Bridge Dept., Dept. of Public Works, Boston, Mass. Refers to J. B. Babcock, 3d, A. W. Dean, G. E. Harkness, A. E. Kleinert, Jr., L. D. Moore, F. C. Pillsbury.

BRUNS, RICHARD PAYNE, Huntington, W. Va. (Age 24). Jun. Engr., U. S. Engr. Office. Refers to M. Blanchard, L. W. Clark, H. C. Corns, F. B. Duis, E. E. Teeter.

BUCK, HORACE MILLER, East Boston, Mass. (Age 34). Job Engr., Boston Traffic Tunnel, Silas Mason Co., Inc., Contrs. Refers to F. Blossom, R. S. Buck, H. H. Haggard, W. E. Hamilton, P. F. Kruse, G. H. Pegram, A. J. Sackett, O. Singstad, H. Smith, H. H. Snyder, A. S. Tuttle.

CURTIS, LUCIEN BLANCHARD, Pelham, N. Y. (Age 23). Refers to G. E. Beggs, F. H. Constant, C. M. Spofford.

DARBY, JESSE MITCHELL, Best, Tex. (Age 23). Field Clerk and Asst. Supt., R. W. Briggs & Co. Refers to E. C. H. Bantel, R. W. Briggs, E. B. Darby, J. A. Focht, R. E. Killmer.

DENNIS, RICHARD WHITING, Los Angeles, Cal. (Age 24). Refers to H. N. Ogden, J. E. Perry.

DOUGLASS, ARDEN HEMAN, Wichita Falls, Tex. (Age 50). Mgr., Water Dept., and Supt. of Sewage Disposal. Refers to E. B. Black, G. D. Fairtrace, O. N. Floyd, J. D. Fowler, H. R. F. Helland, B. B. Hodgman, J. L. Lochridge, A. M. McPherson, J. Montgomery, M. C. Nichols, R. A. Thompson, F. M. Veatch, N. T. Veatch, Jr., J. E. Ward, P. A. Welty.

FARGHER, THOMAS EDWIN NELSON, Loughborough, Leicestershire, England. (Age 40). Head, Dept. of Mech. and Civ. Eng., Loughborough Coll. Refers to R. T. Betts, W. J. E. Binnie, A. Blair, H. J. F. Gourley, R. E. Stradling.

FERNANDEZ CASADO, CARLOS, Madrid, Spain. (Age 27). Cons. Engr. Refers to L. J. Proctor, E. Terradas Ylla. (Applies in accordance with Sec. 1, Art. I, of the By-Laws).

FORBIS, JOHNSON LINVILLE, Roosevelt, Okla. (Age 21). Bridge Inspector, Oklahoma State Highway Dept. Refers to J. F. Brookes, L. M. Bush, N. E. Wolfard.

GATLIN, ROBERT HENRY, Durham, N. C. (Age 23). Instructor in Civ. Eng., Duke Univ. Refers to W. H. Hall, C. L. Mann, H. Tucker.

GRIFFIN, JAMES H., Brooklyn, N. Y. (Age 43). Engr.-in-Chg., Contr. Div., Board of Transportation, New York City. Refers to J. A. A. Connelly, J. W. Daly, E. G. Haines, R. H. Jacobs, A. I. Raisman, R. Ridgway, H. D. Winsor.

HAAS, EDWARD THOMPSON, New York City. (Age 25). Refers to E. F. Haas, F. C. Herrmann, J. P. Hogan.

HOLLOWAY, ALLAN THOMPSON, Wynnewood, Pa. (Age 29). Engr., United Engrs. and Constructors, Inc. Refers to F. O. Dufour, J. T. Kiernan, J. W. May, A. Miedwig, J. W. Moffett, J. L. Orr, L. E. Raymond.

HOLSTEIN, PAUL WHERRITT, Jr., Columbus, Ohio. (Age 22). Draftsman, Div. of Eng. and Constr., City of Columbus. Refers to R. A. Allton, J. H. Blodgett, P. M. Holmes, R. B. Jennings, R. T. Regester.

HOWE, JOSEPH WARNER, Iowa City, Iowa. (Age 30). Asst. Prof. of Mechanics and Hydraulics, State Univ. of Iowa. Refers to A. Davis, M. L. Enger, R. B. Kittredge, F. T. Mavis, F. A. Nagler, C. C. Williams, S. M. Woodward.

JOHNSON, LeROY FRANCIS, Lebanon, N. H., (Age 37). Div. Engr., New Hampshire Highway Dept. Refers to J. W. Childs, F. E. Everett, W. A. Grover, L. C. Marshall, A. E. Page, E. H. Sargent.

LAHDE, WALTER, Ann Arbor, Mich. Age 30). Senior Engr. with Jensen, Bowen & Farrell and H. E. Riggs. Refers to T. L. K. Donnelly, E. L. Eriksen, K. A. Farrell, H. K. Hood, C. E. Jenkins, O. A. R. V. Jensen, H. E. Riggs, W. C. Sadler, J. F. Walker.

LAZAR, DANIEL MORRISON, New York City (Age 25). Engr., Triest Contr. Corporation, Jamaica, N. Y. Refers to H. M. Braloff, I. L. Gelder, J. Meltzer, G. Paaswell, C. J. Robison.

MUNRO, ALEXANDER DUNCAN, Nairobi, Kenya Colony. (Age 39). Engr., Municipality of Nairobi, Kenya Colony. Refers to J. D. Rennie, R. K. Stockwell. (Applies in accordance with Sec. 1, Art. I, of the By-Laws).

SCHINDEL, ROBERT LEE, Jr., Pittsburgh, Pa. (Age 23). Refers to A. Diefendorf, L. C. McCandliss.

SCHINDEL, WILLIAM NEWTON, Pittsburgh, Pa. (Age 22). Refers to A. Diefendorf, L. C. McCandliss.

SCHREWE, ERWIN WERNER, Woodside, N. Y. (Age 50). Topographical Draftsman, Dept. of Water Supply, New York City. Refers to A. J. Bernstein, A. Dick, H. C. Elton, G. A. Gessner, W. H. Kershaw, J. A. McElroy, A. H. Terry.

SPELL, WILLIAM ARTHUR, Atlanta, Ga. (Age 50). Chf. Engr., Atlanta, Birmingham & Coast R. R. Refers to W. A. Hansell, J. A. Higgs, Jr., J. H. Johnston, C. E. Kauffmann, S. B. Slack, S. R. Young.

SPENCE, ELMER LENOX, Philadelphia, Pa. (Age 32). Senior Structural Draftsman, Dept. of City Transit. Refers to J. E. Boatrite, S. Harris, S. S. A. Keast, I. L. Lewis, C. H. Stevens, W. E. Witte.

STUBBS, HENRY FRANKLIN, Seguin, Tex. (Age 38). Gen. Supt. of Eng. Constr., Sumner Sollitt Co. of Texas. Refers to O. N. Floyd, H. L. Fruend, H. S. Hunt, L. H. Huntley, T. C. Shuler.

WARD, RICHARD B, Glendale, Cal. (Age 43). Asst. Engr., Metropolitan Water Dist. of Southern California. Refers to H. R. Bolton, S. A. Kerr, J. S. Longwell, H. K. Palmer, A. M. Rawn, H. E. Warrington.

WARDLE, HARRY KENNETH, Kingston, Jamaica. (Age 46). Civ. Engr. and Archt. Refers to E. N. Bancroft, F. L. Bronstorph, H. J. Dignum, S. C. Henriques, A. A. Simms.

WHITE, ROY ALLAN, Los Angeles, Cal. (Age 48). Res. Engr., Los Angeles County Flood Control Dist. Refers to P. W. Clancy, E. C. Eaton, B. J. Lambert, W. W. Patch, F. Thomas.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

ANDERSON, NORVAL EUGENE, Assoc. M., La Grange, Ill. (Elected March 14, 1927.) (Age 35.) Senior Civ. Engr., Sewage-Treatment Plant Design Div., San. Dist. of Chicago. Refers to L. B. Barker, O. L. Eltinge, I. P. Kane, L. Pearse, H. P. Ramey, H. S. Ripley, L. C. Whittemore.

BARNARD, ARCHER FORTESCUE, Assoc. M., Los Angeles, Cal. (Elected Aug. 31, 1915). (Age 50). Member of firm, Quinton, Code & Hill-Leeds & Barnard, Engrs. Consolidated. Refers to W. K. Barnard, R. K. Brown, H. W. Dennis, W. E. Jessup, J. H. Knowles, O. A. Stone, F. Thomas.

CAMPBELL, BENJAMIN LUCIEN, Assoc. M., Portland, Ore. (Elected Aug. 17, 1931). (Age 49). Draftsman and Asst. Civ. Engr., War Dept., U. S. Engrs. Refers to A. L. Alin, G. R. Lukesh, S. Murray, J. H. Polhemus, H. A. Rands, O. E. Stanley, C. F. Thomas.

EGNER, CARL ALEXANDER, Assoc. M., Norfolk, Va. (Elected July 6, 1920). (Age 38). Hydrographic and Geodetic Engr., U. S. Coast and Geodetic Survey, Washington, D. C. Refers to F. S. Borden, W. Bowie, H. A. Cotton, C. L. Garner, R. R. Lukens, E. H. Pagenhart, W. E. Parker.

HOOD, HUGH KENDALL, Assoc. M., Ann Arbor, Mich. (Elected Feb. 1, 1910). (Age 51). Senior Engr. with Jensen, Bowen & Farrell and H. E. Riggs. Refers to T. L. K. Donnelly, E. L. Eriksen, K. A. Farrell, B. H. Hardaway, Jr., R. E. Hardaway, Jr., O. A. R. V. Jensen, H. E. Riggs, W. C. Sadler, E. L. Scruggs.

NEGREY, STEPHEN, Assoc. M., Elizabeth, N. J. (Elected Oct. 12, 1925). (Age 38). Gen. Mgr. and Treas., Masem Constr. Co., Inc., Gen. Bldg. Contrs., Brooklyn, N. Y. Refers to A. F. Callahan, H. J. Carroll, M. C. Cleveland, T. E. Collins, E. C. Epple, W. C. Rohdenburg, T. K. Thomson.

SPEIR, OSWALD, Assoc. M., Redwood City, Cal. (Elected Junior Oct. 14, 1919; Assoc. M. April 3, 1922). (Age 40). Engr. Refers to E. K. Barnum, B. A. Etcheverry, H. Forbes, J. D. Galloway, H. F. Gray, L. R. Jorgensen, M. M. O'Shaughnessy, T. H. Wiggin.

WILHELM, FREDERICK EDWARD, Assoc. M., Denver, Colo. (Elected May 19, 1924.) (Age 40). Associate Engr., U. S. Bureau of Reclamation. Refers to S. O. Harper, C. S. Lambie, J. L. Savage, R. G. Shankland, C. Voetsch, R. F. Walter, R. D. Welsh.

FROM THE GRADE OF JUNIOR

BROWN, DALTON MUNROE, Jun., Brooklyn, N. Y. (Elected April 18, 1927.) (Age 32). Engr., Marcus Contr. Co. Refers to F. N. Benedict, F. E. Cudworth, S. T. Goldsmith, H. M. Hale, L. S. Joseph, H. W. Lesh, A. P. Richmond, Jr., L. White.

COVAS, PERFECTO ANTONIO, Jun., Rochester, N. Y. (Elected Jan. 13, 1930) (Age 32). Computer, Div. of Design and Heavy Constr. Refers to H. W. Baker, E. A. Fisher, I. E. Matthews, C. A. Poole, W. H. Roberts, J. F. Skinner, E. H. Walker.

KELLY, EUGENE THOMAS, Jun., Forest Hills, N. Y. (Elected April 15, 1929). (Age 32). Jun. Engr., New York & Queens Elec. Light & Power Co., Flushing, N. Y. Refers to F. W. Allen, G. A. Brinkerhoff, W. J. McGrath, W. A. Mellny, G. Perrine, W. H. Walker.

MATTHEWS, JOHN HORACE, Jun., West Lafayette, Ind. (Elected Oct. 10, 1927). (Age 30). Instructor in Civ. Eng., Purdue Univ. Refers to E. L. Eriksen, J. E. Hall, C. A. Hart, W. K. Hatt, W. J. Henderson, S. C. Hollister, R. B. Wiley.

THOMPSON, JOHN STANLEY, Jun., Hudson, N. Y. (Elected Aug. 28, 1922). (Age 32). Pres. and Treas., The Thompson Constr. Corporation. Refers to C. M. Africa, R. H. Chambers, E. S. Jarrett, T. R. Lawson, P. C. Ricketts.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.

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* Director John R. Slattery died September 23, 1932.

AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

BOARD OF DIRECTION MEETINGS

January 16-17, 1933:
A Quarterly Meeting will be held at New York, N. Y.

ANNUAL MEETING

January 18, 19, 20, and 21, 1933

NEW YORK, N. Y.

January 18, 1933:

Morning.—Annual Meeting. Conferring of Honorary Membership, and Presentation of Medals and Prizes.
Afternoon.—General Meeting.
Evening.—President's and Honorary Members' Reception and Dinner.

January 19, 1933:

Morning.—Technical Division Sessions.
Afternoon.—Technical Division Sessions.
Evening.—Entertainment and Smoker.

January 20, 1933:

All-Day Excursion to West Point Military Academy and Bear Mountain Park.

January 21, 1933:

Morning.—Inspection Trips.

The Reading Room of the Society is open from 9:00 A. M. to 5:00 P. M. every day, except Sundays and holidays; from May to September, inclusive, it is closed on Saturdays at 12:00 M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 274 current periodicals, the latest technical books, and the room is well supplied with writing tables.

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